Validation of wave response analysis for jack-up rigs

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For the Health and Safety Executive

OFFSHORE TECHNOLOGY REPORT
2000/094

First published 2000

ISBN 0 7176 1902 8

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1 ABSTRACT

For dynamically sensitive systems such as jack-up rigs, a dynamic analysis is performed to assess their structural response. In order of their increasing complexity, the available analysis methods may be broadly classified as: (1) Single degree of freedom; (2) Frequency domain; and (3) Time domain approach. Some of these analysis techniques are outlined in SNAME T&R 5-5A.

The random time domain approach is considered most accurate although most complex, whereas the SDOF method is the simplest but approximate. Frequency domain methods are generally applied to simple linearized models. A frequency decomposition of the wave forces on a structure indicates that even for a regular wave, higher harmonic components are present due to non-linearities. Therefore, SDOF method severely overestimates the inertia forces near the natural period, while possibly underestimating the dynamic response further away.

This study recommends a practical frequency-domain dynamic analysis method referred here as "Wave Response Analysis" (WAVRES), which leads to comparable results on extreme values as the random time domain analysis. For regular wave steadystate response, the method is based on mode superposition without the assumption of linearization and considers the effect of higher harmonics, which reduces the overall resonance effects predicted by the SDOF method. This method is further extended to random seas. It is computationally very efficient, applicable to reasonably large size models and amenable to simple interpretation. Therefore, this method is seen as a close alternative to the random time domain analysis, and offers significant time saving to the practicing engineer without sacrificing the accuracy and being unduly conservative. Nonlinear effects such as P-delta effect, foundation stiffness etc. can be included in the method through suitable linearization.

The purpose of the study is to obtain the results from the proposed method and to compare them to more accurate time domain methods. The study outlines the proposed technique, and discusses the various frequency domain methods, such as drag-inertia parameter and Rayleigh distribution. In order to benchmark this method, the study analyzes a few practical examples of jack-up rig models and compares results from the above analyses with the time domain results for both regular and irregular waves. It has been demonstrated that "Wave Response Analysis" (WAVRES), leads to comparable results of extreme values of dynamic response as the random time domain analysis. The method appears to give good results irrespective of the jack-up model, water depth, wave heights and periods, and foundation fixity conditions. For high current, static response using drag-inertia parameter appears to be over-predicted, but prediction of inertia forces still remains accurate, which is of prime importance for design assessment.

2 INTRODUCTION

For dynamically sensitive structures such as jack-up rigs, compliant towers etc. the performance of dynamic analysis of some sort is almost inevitable to assess the safety of the unit during design storms. The dynamic analysis model becomes complicated because of several reasons. The primary ones among these are: non-linearity of the drag force, random nature of the wave, and structural or foundation non-linearity. In this paper we exclude the material and structural non-linearity factors but include discussions on the nature of drag forces, the random process involved and the prediction of maximum expected response.

The current practice of analyzing such systems varies over a wide range. It starts with the simplest method of idealizing the structure as a single degree of freedom system (SDOF) for the evaluation of the dynamic amplification factor (DAF), to the most accurate one of nonlinear time domain method using random waves. Moreover, for the time domain simulation, since generally only one time series is realized, statistical extrapolation is absolutely necessary to extract meaningful results. In view of these limitations, the present paper discusses a frequency-domain method that is less complex and computationally less intensive than the time domain solution, but produces results much better than the SDOF method and often close to the random time domain method.

In the next few sections, the frequency-domain method is elaborated. First, the method is described for regular waves in order to bring out the essential features of drag loading and its implication to the DAF evaluation. Then it is extended to include a wave spectral description, along with procedures for statistical extreme value prediction. The method is applied to the following examples of jack-up rigs and the results are compared to similar results obtained from SDOF modeling and random time domain simulation:

Example 1	a) LeTourneau's 116-C Model – 300 ft water depth
	b) LeTourneau's 116-C Model – 250 ft water depth
Example 2	Friede & Goldman L-780 Model – 300 ft water depth

The method is a practical tool that can be used for complex structural geometry to obtain meaningful results, but may not possess very strict mathematical justification.

3 WAVE RESPONSE ANALYSIS

The wave response analysis and its extension to spectral analysis has been described in detail in a recent paper, “A Practical Frequency-Domain Method for Random-Wave Analysis and Its Application to Jack-up Units”, by Partha Chakrabarti, et al., OTC 10795, May 1999. A summary of this method is presented here.

3.1 HYDRODYNAMIC ANALYSIS

For structures of interest here, the members are slender and prismatic. The wave forces on this type of members are calculated by Morison equation. The total wave force is given by the summation of a drag component and an inertia component. The drag force depends on the square of the fluid particle velocity and introduces nonlinearity to the excitation force. The higher order terms in the dynamic analysis occur not only because of the velocity-squared term but also due to higher order wave theory, variable submergence of elements near the water surface, etc.

For most frequency domain methods linearization is necessary for the solution of the equations of motion (Gudmestad et al, 1983), in which higher terms are dropped for mathematical simplification. Usually the equivalent linearized drag and inertia components are evaluated and used for wave load computation and subsequent structural dynamic analysis.

As will be explained later, wave response method does not need any drag linearization.

3.2 STRUCTURAL ANALYSIS

The equation of motion of a structure can be written in terms of the structural displacement, v as:

$$M_o \ddot{v}(t) + C \dot{v}(t) + Kv(t) = P(t) \quad \text{... 1}$$

In general the above (Eq. 1) is a matrix equation, where M_o is the mass matrix which includes added mass effects for submerged members, *C* is the damping matrix, *K* is the linear stiffness matrix, and *P(t)* is the external load vector.

For linear systems the above equations can be decoupled using mode shapes and generalized coordinates in placement of the structural displacements etc. For the uncoupled set of equations, the response for a steadystate harmonic excitation is harmonic, and it is fairly simple to express the solution in closed form. However, structures such as jack-ups experience large drag force. Therefore, even in regular waves, the hydrodynamic forcing function would include harmonics higher than the wave frequency as well as a steady force such as, due to current. Thus the contribution to the total force on the structure is distributed over a range of discrete harmonics.

This is demonstrated in Figs. 3-1 and 3-2. In Fig. 3-1, the total static and dynamic base shear, on a jack-up rig structural model is plotted as time history over one wave period. Fig. 3-2 plots the Fourier transform coefficients for the base shear. These two plots clearly show that the wave force time history is not purely sinusoidal, but has higher harmonics present.

Fig. 3-1 Base Shear Time History

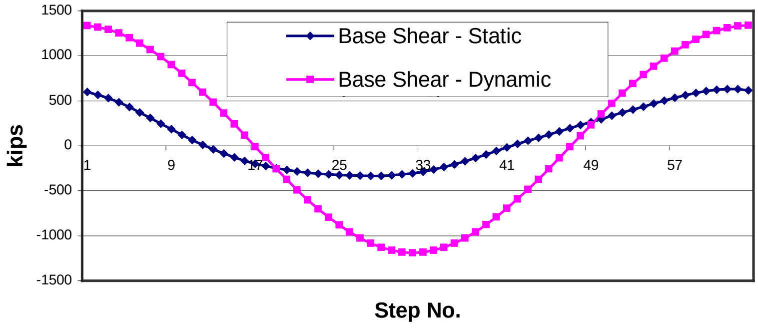
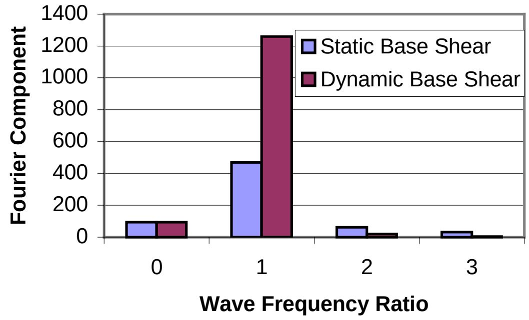


Fig. 3-2 Fourier Components in Wave Force



Irregular seastate is usually represented by a wave spectrum, which can be viewed as a superposition of harmonic waves of different frequencies, amplitudes and phases. Due to the nonlinearity of the drag force, frequency domain approaches normally fail to represent the true statistical properties of excitation and response unless special simplifications are made. Therefore, a usual method is to perform time domain calculations for the structural response. In this method, the wave train is simulated by the superposition of a large number of Airy waves of different frequencies, the amplitudes of which are selected from a description of the wave spectrum and phases are selected as random variables. Care is taken so that the resulting wave train has the same statistical properties as the original spectrum. Such time domain calculations should be performed for a sufficiently long duration and several realizations have to be analyzed to achieve stable statistics of response. SNAME TR-5-5A (1997) has recommendations concerning such dynamic analysis for site assessment of jack-up rigs.

3.3 THE WAVE RESPONSE METHOD

The procedure for wave response method of analysis is described here, first for regular waves and is then extended to irregular waves.

3.3.1 Steady State Response to Regular Waves

In the wave response method of analysis proposed here, the force is not linearized, but rather the hydrodynamic excitation is expressed as a superposition of several harmonics. For regular waves, the following steps are used:

- A linear or appropriate higher order wave theory is used and the wave force on the structure is calculated stepping the wave one wavelength through the structure in small increments.
- As many eigenmodes of the structure as are required to have the desired mass participation are extracted.
- Fourier components of the generalized loading for each mode are evaluated. In general several higher harmonics which are multiples of wave frequency will be present. The number of Fourier components is chosen based on the accuracy of the representation desired.
- Steadystate modal responses to these individual Fourier components due to the selected modes are evaluated.

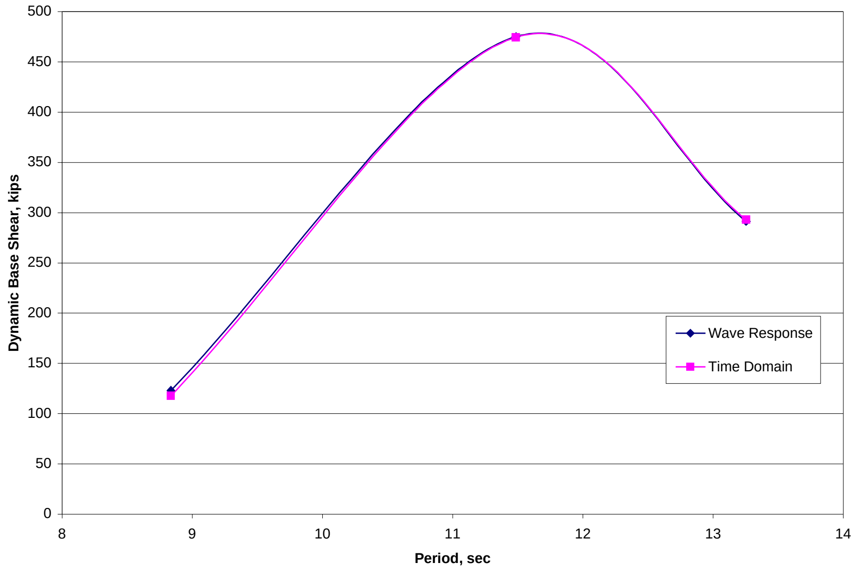
For the final structural analysis, modes are superposed for the dynamic response with due regard to phase relationships of all the components

Since the method depends on modal superposition, the structural system should essentially be linear. In certain cases involving limited nonlinear elements such as the foundations, linearized properties can be used in the structural model. Other improvements such as isolating static and dynamic response for enhanced recovery of the static response can also be achieved. For regular waves, the method produces results almost identical to the time domain analysis. This is demonstrated in Table 3-1 and Fig. 3-3, where the dynamic response of a jack-up model is analyzed for three wave periods both using the wave response method and time domain analysis. The maximum amplitude of dynamic base shear response is plotted for comparison. As the plot indicates, the results from both the procedures match almost exactly. The small variation noticed is not due to the method, but due to a slight difference in segmentation used for wave force calculation on a member.

Table 3-1 Comparison of Wave Response and Time Domain Regular Wave Analysis

Period, sec	Static Base Shear, kips		Dynamic Base Shear, kips	
	Wave Response	Time Domain	Wave Response	Time Domain
8.836	112.5	106.77	123.12	117.78
11.486	136.54	132.47	475.17	474.47
13.254	145.67	142.27	291.25	292.98

Fig. 3-3 Validation of Regular Wave Response withTime Domain



3.3.2 Response to Irregular Waves

For irregular waves, the sea state is represented by a wave spectrum $S_\eta(\omega)$. Spectral analysis for any linear system involves constructing the transfer function of the response $R(\omega)$, such as base shear, lateral deflection, etc. and using the following expression for the response spectrum $S_R(\omega)$

$$S_R(\omega) = S_\eta(\omega) \cdot R^2(\omega) \qquad \text{... 2}$$

The wave spectrum is normally given for a site, and often, standard forms of the spectrum such as Pierson-Moskowitz, JONSWAP etc. are used. For dynamic wave analysis, the construction of the structural transfer function will involve analyzing the structure for a range of frequencies at unit wave amplitude. For a zero-mean Gaussian process, once the response spectrum is evaluated, the statistical properties of the response, such as the standard deviation, zero-crossing period etc. can be found by standard spectral analysis techniques. Extreme values are predicted assuming the peaks to be Rayleigh distributed. However, when the process involves nonlinearity such as due to drag, the evaluation of response and its statistics is not straightforward. The difficulties arise because

- The transfer function itself is not linear, i.e., it is dependent on the height of the regular wave selected
- The process is not Gaussian, and the evaluation of statistical extreme is more complex.

In order to address the above difficulties several methods have been tried in the past. These methods with some modifications are discussed briefly.

3.3.3 Selection of Wave Height-Period for Transfer Function

As mentioned earlier, this selection is very important for subsequent response calculations in view of the nonlinear nature of drag forces. Many references [e.g., 3] suggest a relationship of wave period of the form:

$$T = 1 + 4.1 H_{\det}^{0.4} \qquad \text{... 3}$$

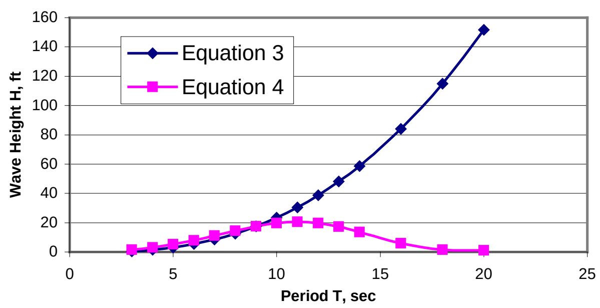
where $H_{det} = 0.86H_{max}$, and H is in meters and T in seconds. The relationship implies that the wave height increases almost parabolically with the period.

In this paper an alternate choice of wave height-period relationship is proposed for each incremental T_i ($\omega_i=2\pi/T_i$) selected for constructing the transfer function. The relationship proposed here is based on the wave energy distribution and is of the form:

$$H_i = H_s \sqrt{\frac{S_i(\omega)}{S_o(\omega_o)}} \qquad \text{... 4}$$

where H_s is the significant wave height and ω_o is the frequency corresponding to the peak period. The advantage of the above relationship is that it maintains the relative importance of various frequencies in a random wave spectrum, Fig. 3-4.

Fig. 3-4 Period to Height Relationship



It has been demonstrated in an earlier work [1] that transfer functions using wave height-period relationship given by the traditional formula (Eq. 3) could overestimate the response especially at higher periods. Transfer function using wave heights in proposed method (Eq. 4) can predict the responses much better. Therefore, this method has been adopted.

3.3.4 Maximum Value Prediction

For a Gaussian system, the usual assumption involved for predicting the maximum value is to use Rayleigh distribution. For a non-Gaussian process, other assumptions are necessary in predicting the most probable maximum response, as Rayleigh distribution leads to underestimation. Application of drag-inertia parameter method of prediction will be discussed.

Rayleigh Distribution. For Gaussian process, the most probable maximum response mpm_R is given by the following well-known expression:

$$mpm_R = \sigma_R \sqrt{2 \cdot \ln(N)} = \sigma_R C_R \qquad \text{... 5}$$

where σ_R is the standard deviation of response R and N is the number of cycles of the response based on the storm duration. For example, considering a storm duration of 3 hours, and a zero-crossing period of the response equal to 10 seconds, N=1080 and the multiplication factor C_R becomes 3.73

Drag-Inertia Parameter Approach. The drag-inertia parameter is defined as the ratio of the magnitude of the drag force to the magnitude of the inertia force, due to waves. In the absence of current for an element of a circular cylinder of diameter D and unit length, subjected to a periodic wave, the drag-inertia parameter becomes:

$$K = \frac{2 C_D u^2}{\pi C_M D \dot{u}} \qquad \text{... 6}$$

As both u and $\dot{u}$ depend on the wave parameter (wave height, period, and water depth), and the elevation at which the element is located, K is also a function of these parameters. Therefore, theoretically, K is only meaningful for Morison wave loading for a unit length of the member.

The definition of K can be extended to random waves by replacing the deterministic normal velocity and acceleration by the standard deviation of the random velocity and acceleration respectively. Thus for a single element:

$$K = \sqrt{\frac{\pi}{8}} \frac{\sigma_R(C_M = 0)}{\sigma_R(C_D = 0)} \qquad \text{... 7}$$

where the variable R corresponds to the wave force per unit length in a random sea. In the above expression, the terms in the parenthesis mean that the quantity is evaluated by setting one of the hydrodynamic parameters to zero.

The above equation can be generalized to apply to any local or global response R of interest. This generalization is purely an engineering postulate and does not have a strict theoretical foundation.

The mpm-factor C_R, which is the ratio of the most probable maximum value to the standard deviation is given as a function of the drag-inertia parameter K. For drag dominated structures ($C_M=0$), C_R=8.0 and for inertia dominated structures, ($C_D=0$), C_R=3.7.

Hybrid Method. Earlier studies [2] indicated that the drag-inertia approach gives better results for static wave force maximum than Rayleigh distribution, although it overpredicts compared to the time domain method. On the other hand, it underpredicts the dynamic response significantly, whereas Rayleigh distribution matches the time domain results quite closely. This is expected since the dynamic inertia force on a structure arises from its vibrations, and for a linear structure, the inertia force is expected to closely follow a Gaussian process.

In view of the above it was concluded that a hybrid method where the static maximum response is computed using drag-inertia approach and the extreme dynamic inertia force from Rayleigh distribution would produce better results.

This hybrid method has been implemented in the wave response methodology.

3.3.5 Design Force

In wave response methodology, the total inertia force (or overturning moment) on the structure is calculated using the spectral analysis technique described above using Rayleigh distribution. In a subsequent pass of the analysis, the structure is analyzed using the regular wave response method for a wave with H_{det} and T_p. The inertia forces are then scaled to match the base shear obtained from Rayleigh distribution. This way, the overall distribution of the inertia force on the structure is retained. This is particularly convenient because, for structural analysis one is interested in not only the base shear but overall performance of the structure including evaluation of joint deflections, member stresses etc., which is difficult in a probabilistic format. Thus the total design force is static wave force for a regular wave plus the inertia force from spectral analysis, similar in concept to that recommended in [3]. One may chose to position the structure with respect to the wave phase such that either maximum base shear or overturning moment become the governing criteria for selection. It should be noted that the drag-inertia prediction of static wave force maximum is only required to calculate the DAF, it is not required to calculate the design force.

4 EXAMPLE PROBLEMS

A jack-up rig is a dynamically sensitive structure specially when it operates in deeper water. In order to demonstrate the relative merits of the various techniques of analysis described, two example structures of independent latticed leg jack-up rig have been used. These are:

Example 1	a) LeTourneau's 116-C Model – 300 ft water depth
	b) LeTourneau's 116-C Model – 250 ft water depth
Example 2	Friede & Goldman L-780 Model – 300 ft water depth

For each of the examples two cases are considered: (i) pinned base, (ii) spudcan with fixity. Thus the cases analyzed represent a wide range of dynamic system.

The structural models use simple beams and three stick legs. The models have been calibrated for stiffness, wave loads and dynamics with more detailed plate-beam models. For equivalent latticed leg segments hydrodynamic parameters, C_D and C_M, have been evaluated based on procedures given in [3]. To include P-delta effects, negative horizontal springs have been included at the hull level.

The behavior of spudcan is nonlinear due to the soil-structure interaction. For case (ii), spudcan springs are applied for all three translational and two horizontal rotational directions. These springs represent linearized springs for the non-linear spudcan behavior. It is assumed that the global inertial load on the structure can be well predicted using a time-averaged spring stiffness of the foundation, as long as soil failure does not occur.

A total of 10 modes have been used for all calculations. The damping used is 7% of critical for all modes.

All calculations for time-domain as well as frequency-domain wave response analysis are performed using the software StruCAD*3D [4].

5 LETOURNEAU'S 116-C MODEL – 300 FT WATER DEPTH

The particulars of the 3-leg jack-up model are shown in Table 5-1.

Table 5-1 Details of 116-C Jack-up Rig – 300' Depth

Leg length	477 ft
Air Gap	50 ft
Spud can penetration	25 ft
Support Location	10ft below mudline
Elevated weight	15,100 kips
Leg weight	2683 kips each
Spud can weight	362 kips each
Natural Period - Pinned	10.88 sec
Natural Period – Fixity	7.31 sec
Equivalent Diameter	87.29 in
Drag Coefficient, C_D	4.92
Inertia Coefficient, C_M	2.0

5.1 RESPONSE FOR REGULAR WAVES

To demonstrate the effect of higher harmonics to the dynamic response, the structure with pinned bottom is analyzed using the frequency-domain wave response method for a regular wave of height 33.1ft and a range of periods. A small current of 0.7 knots has been included in the wave model.

Time domain analyses have also been carried out for the same wave periods. Dynamic Amplification Factors (DAFs) obtained from the analyses have been plotted in Fig. 5-1 versus the ratio of natural period to the wave period, T_n/T_w. The corresponding DAFs for a SDOF system are also plotted.

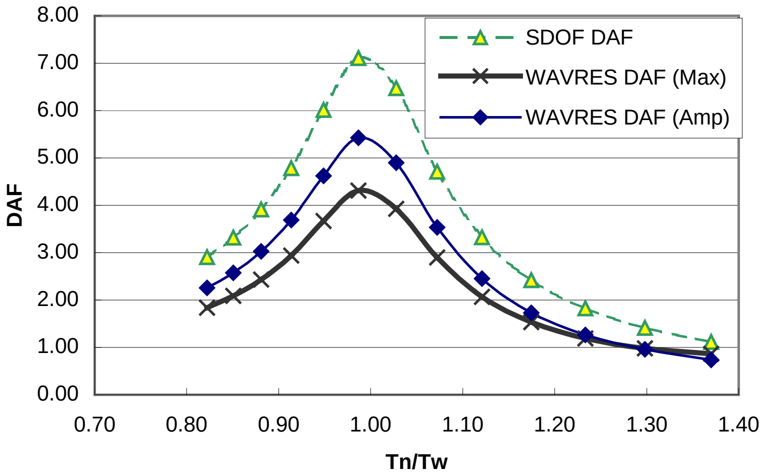
Some explanation of the definition of DAF for wave loads is pertinent here. The pseudo-static variation of the total wave load is calculated by stepping the wave over one wave-period through the structure. As shown before, even for a regular wave, the wave load time-history, including nonlinear effects, shows non-zero mean value. This value is higher when current is present. The DAF, therefore, could be defined in various ways. Two definitions of DAF that are used here are:

$$DAF_1 = \frac{Rdyn_{max} + Rdyn_{min}}{Rstat_{max} + Rstat_{min}} \qquad \text{... 8}$$

$$DAF_2 = \frac{Rdyn_{max}}{Rstat_{max}} \qquad \text{... 9}$$

DAF₁ should be compared with the SDOF DAF, whereas DAF_2 should be compared with the time domain. In Fig. 5-1 the values of DAF_1 (amplitude based) and DAF_2 (maximum value based) have been plotted, which indicate that the DAFs from wave response analysis are much less than the SDOF DAFs because of higher harmonics present in the wave force. One may note that as the wave period moves away from the natural period on either side, the regular wave analysis can even lead to a deamplification of the static response.

Fig. 5-1 Comparison of DAF for Regular Waves



5.2 RESPONSE FOR IRREGULAR WAVES

5.2.1 Pinned Case

Using the transfer function for base shear, the response in irregular waves for a P-M spectrum with H_s=20.69 ft has been evaluated for a range of peak wave periods. For the spectral wave response method, the extreme value calculation for the static wave force has been performed using Drag-Inertia parameter approach. The maximum inertia force, which is defined as the difference between the dynamic and the static mpme values, has been calculated using Rayleigh distribution. The total dynamic response is a sum of the static mpme value and the maximum inertia force. The DAF has been evaluated based on Eq. 9. The design force is the summation of static base shear obtained from a final pass of the regular wave response analysis for H_{det} and T_p plus the inertia force scaled down to the maximum value obtained from spectral analysis.

Similarly, random time domain analysis for the same linear structure has been carried out. For random time domain analysis, Wheeler's form (1969) of Airy waves has been used. For the time domain analysis, five different time history realizations each of 60 minutes duration were analyzed. The most probable maximum value has been evaluated using Weibull distribution for each of the response time histories. The final most probable extreme (mpme) values are obtained as the average ensemble value of the individual sample maximums.

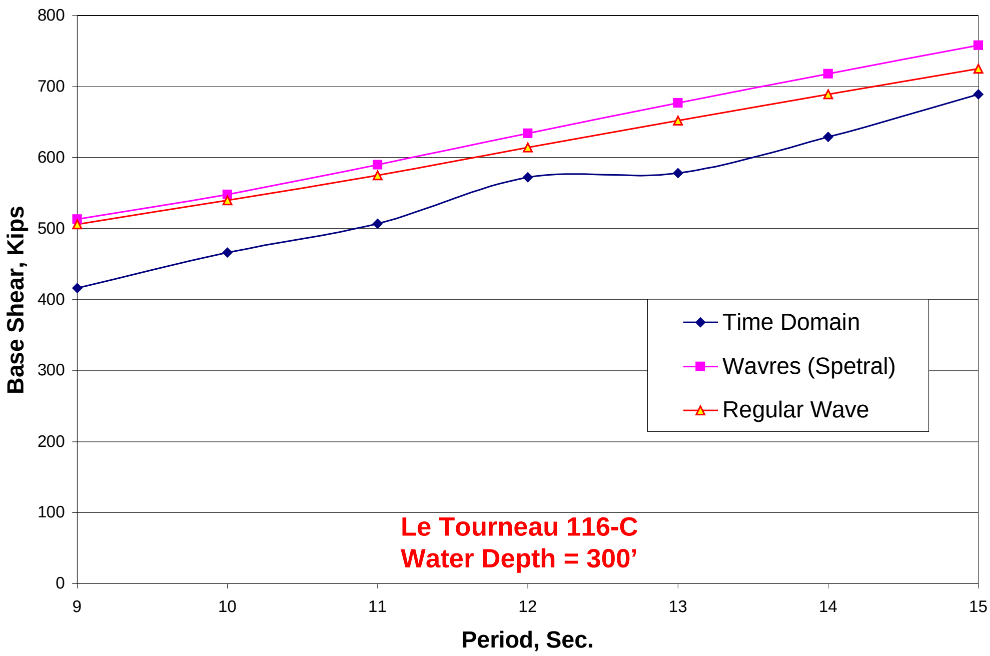
A comparison of the static wave force is made in Table 5-2. The results are also plotted in Fig. 5-2. Results of static wave force obtained from equivalent regular waves using Dean's Stream Function theory have also been included for comparison. The results indicate that maximum static wave force from spectral wave response based on Drag-Inertia method closely match those from a regular static analysis, wave response values being slightly higher

(1-5%). The K factor (Eq. 7) ranges from 0.53-0.73. The random time domain analysis underestimates wave forces by as much as 5-18% from the regular static for almost the entire period range.

Table 5-2 Comparison of Static Base Shear

Tp Sec	Regular Static Base Shear kips	Random Time Domain		Spectral Wave Response	
		Base Shear kips	% Diff.	Base Shear kips	% Diff.
9	506	416	-17.79%	513	1.38%
10	540	466	-13.70%	548	1.48%
11	575	507	-11.83%	590	2.61%
12	614	572	-6.84%	634	3.26%
13	652	578	-11.35%	677	3.83%
14	689	629	-8.71%	718	4.21%
15	725	689	-4.97%	758	4.55%

Fig 5-2 Comparison of Static Base Shear



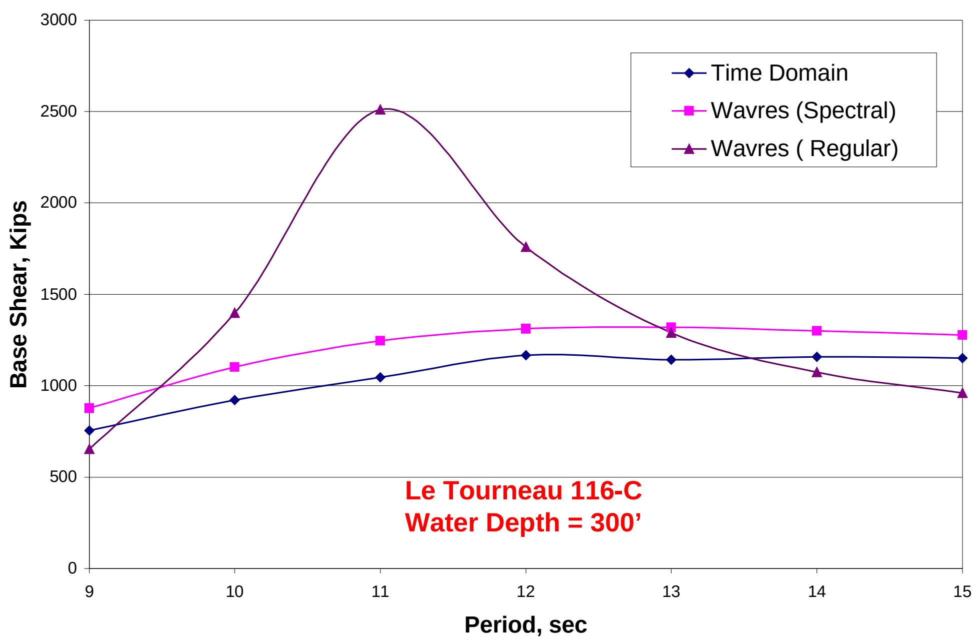
Most probable maximum dynamic response was also computed by spectral wave response and time domain analysis. The maximum inertia force, which is defined as the difference between the dynamic and the static mpme values, is shown in Table 5-3.

Table 5-3 Comparison of Inertia Force

Tp Sec	Regular Wave Response kips	Random Time Domain		Spectral Wave Response		
		Base Shear mpme, kips	Inertia Force kips	Base Shear mpme, kips	Inertia Force kips	Inertia Force % Diff.
9	653	755	339	877	364	7.37%
10	1398	921	455	1102	554	21.76%
11	2511	1045	538	1245	654	21.56%
12	1759	1167	595	1311	676	13.61%
13	1290	1142	564	1320	643	14.01%
14	1074	1158	529	1301	579	9.45%
15	961	1152	463	1278	509	9.94%

The total dynamic base shear from the wave response and random time domain analysis is plotted in Fig. 5-3 and the inertia force plotted in Fig. 5-4. The results from regular wave response analyses are also plotted for comparison. The plots indicate that the inertia force from spectral wave response match the time domain results quite closely, within 7-22%. The regular wave response shows the traditional peak near the natural period and overestimates the response near it. For wave periods about 17-25% (14 sec <T_p<9 sec,) away from the natural period, the regular wave analyses underestimate the dynamic response. This is due to the fact that for regular waves, all energy is still contained at the wave period, which is away from the natural period. Whereas, for a spectral analysis, the tail of the wave spectrum could still overlap the natural period, which increases the dynamic response for irregular wave analysis.

Fig 5-3 Comparison of Total Base Shear - Pinned



The DAF computed using Eq. 9 is shown in Table 5-4 and plotted in Fig. 5-5. Again the spectral wave response results show excellent comparison with time domain. The DAFs from regular wave analyses are also plotted. The spectral wave response results are within +1 to -6% of the time domain results.

Fig 5-4 Comparison of Inertia Force - Pinned

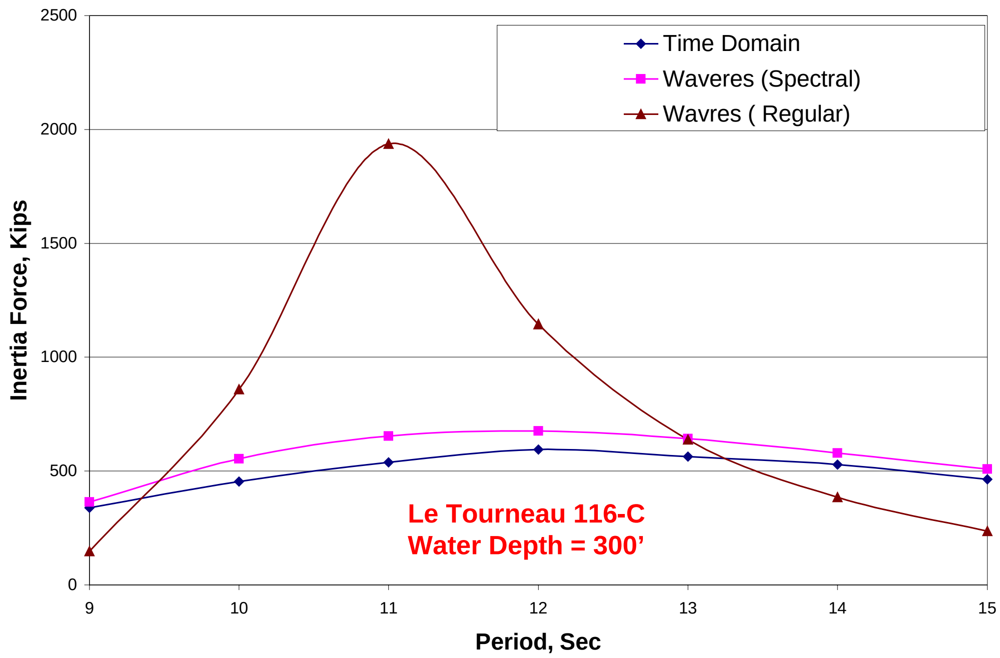
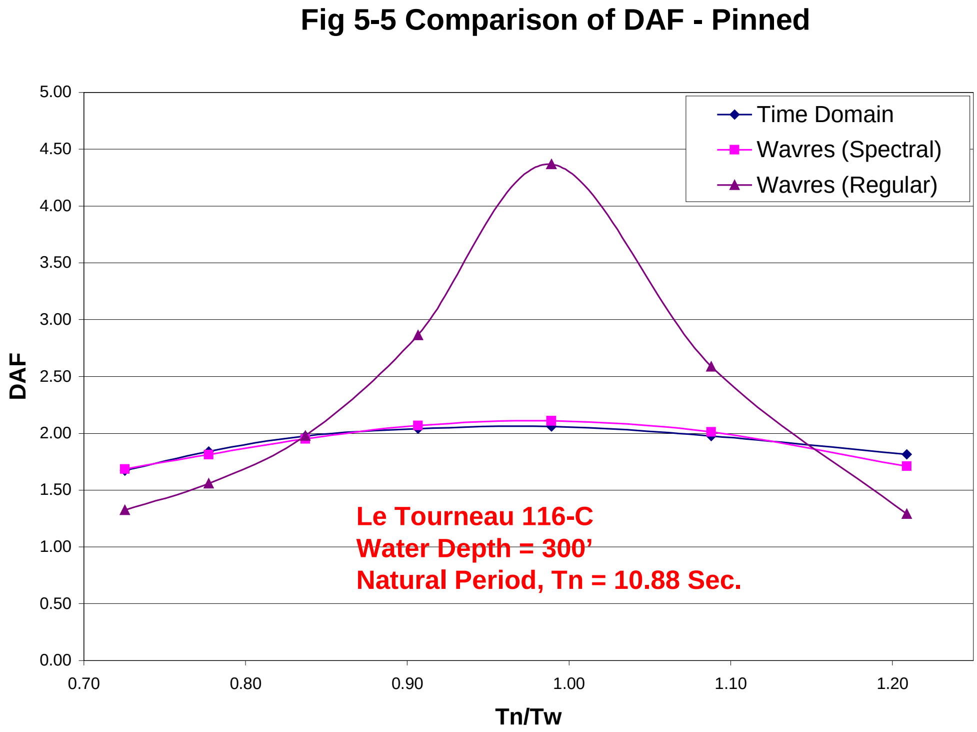


Table 5-4 Comparison of DAF

Tp Sec	Tn/Tw	Regular Wave Response	Random Time Domain	Spectral Wave Response	
			DAF	DAF	% Diff.
9	1.21	1.29	1.81	1.71	-5.52%
10	1.09	2.59	1.98	2.01	2.43%
11	0.99	4.37	2.06	2.11	2.43%
12	0.91	2.86	2.04	2.07	1.47%
13	0.84	1.98	1.98	1.95	-1.52%
14	0.78	1.56	1.84	1.81	-1.63%
15	0.73	1.33	1.67	1.69	1.20%



The design force comprising of regular static and inertia force from spectral calculations are shown in Table 5-5 and plotted in Fig. 5-6. This shows that the spectral wave response follows a smooth trend over the period ranges similar to the random time domain, only the forces are about 3-10% higher.

Table 5-5 Comparison of Design Force

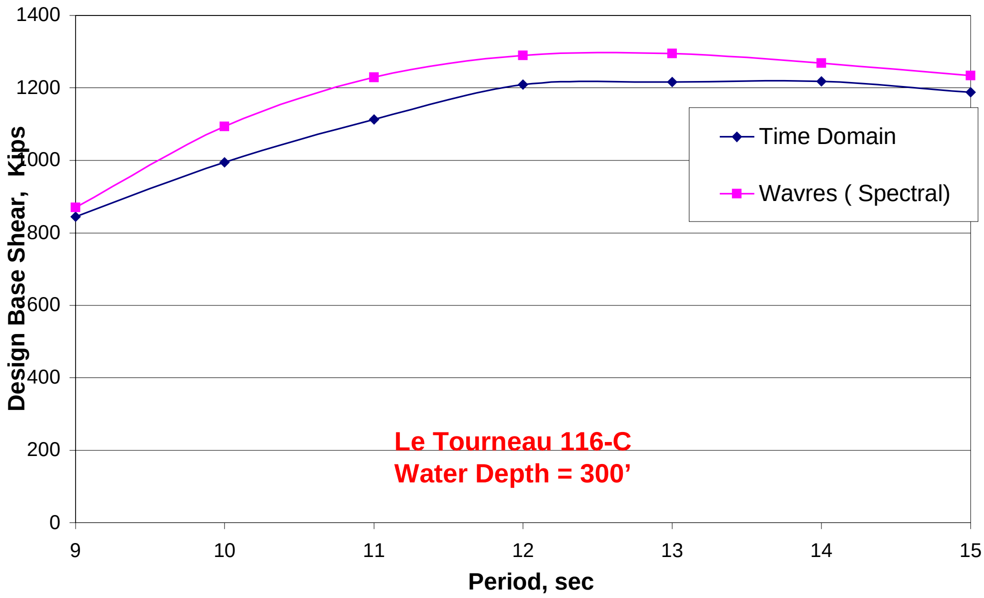
Tp Sec	Random Time Domain	Spectral Wave Response	
	Base Shear kip	Base Shear kip	% Diff.
9	845	870	2.96%
10	995	1094	9.95%
11	1113	1229	10.42%
12	1209	1290	6.70%
13	1216	1295	6.50%
14	1218	1268	4.11%
15	1188	1234	3.87%

The corresponding design overturning moments at the base of the spudcan are tabulated in Table 5-6 and plotted in Fig. 5-7.

Table 5-6 Comparison of Design OTM

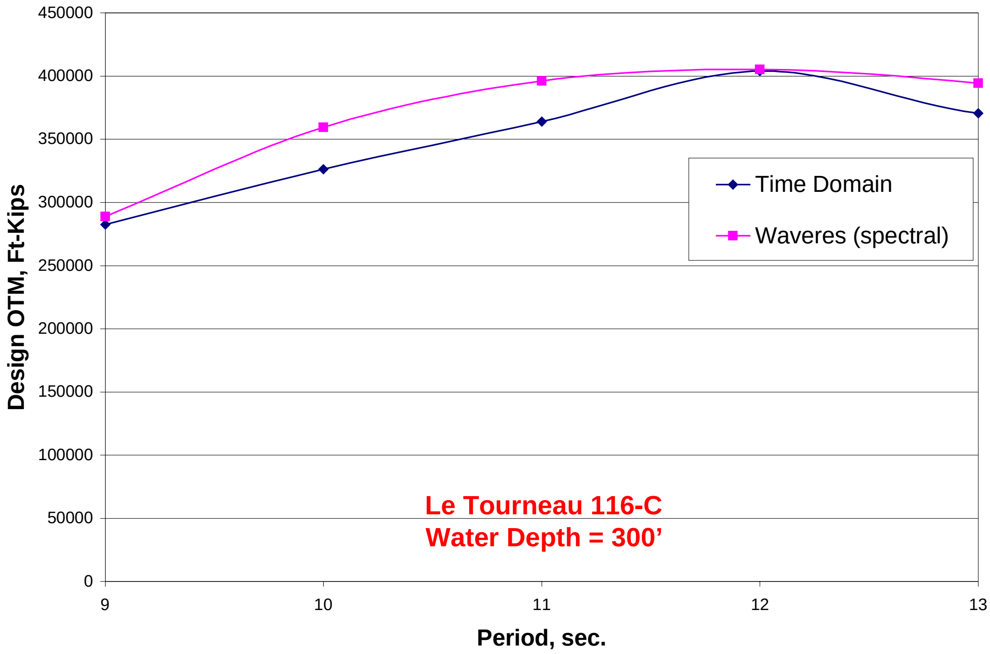
Tp Sec	Random Time Domain	Spectral Wave Response	
	OTM kip-ft	OTM kip-ft	% Diff.
9	282430	288894	2.29%
10	326181	359395	10.18%
11	364046	396115	8.81%
12	403990	405267	0.32%
13	370620	394371	6.41%
14	361271	373225	3.31%
15	342631	351167	2.49%

Fig 5-6 Comparison of Design Force - Pinned



The design OTM using spectral wave response is within 0-10% of the time domain results.

Fig 5-7 Comparison of Design OTM - Pinned



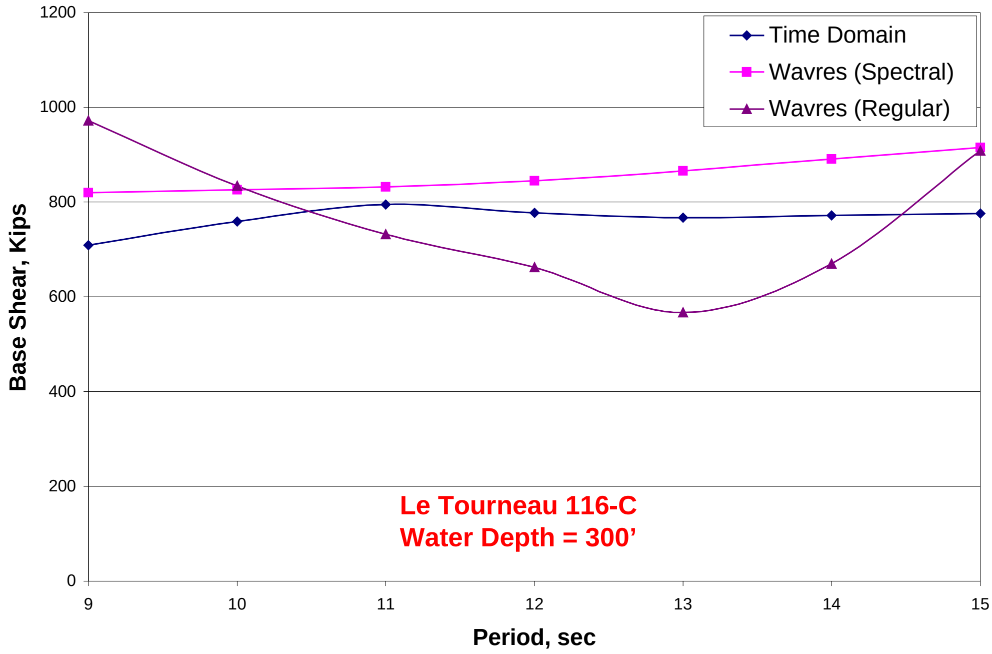
5.2.2 Fixity Case

Similar analyses have been performed for spud cans with fixity. The static wave force is the same for the pinned condition. Most probable maximum dynamic response was computed by spectral wave response and time domain analysis. The maximum inertia force is shown in Table 5-7.

Table 5-7 Comparison of Inertia Force

Tp Sec	Regular Wave Response kips	Random Time Domain		Spectral Wave Response		
		Base Shear mpme, kips	Inertia Force kips	Base Shear mpme, kips	Inertia Force kips	Inertia Force % Diff.
9	972	709	293	820	307	4.78%
10	834	759	293	826	278	-5.12%
11	732	795	288	832	242	-15.97%
12	662	777	205	845	211	2.93%
13	567	767	189	866	189	0.00%
14	670	772	143	891	173	20.98%
15	909	776	87	915	157	80.46%

Fig 5-8 Comparison of Total Base Shear - Fixity



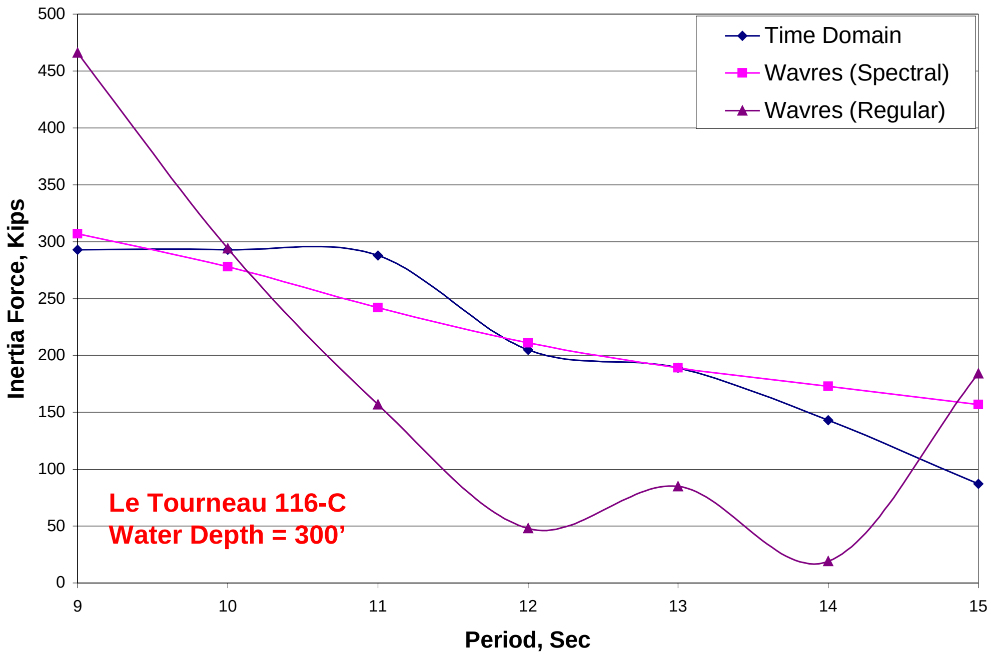
The total dynamic base shear from the wave response and random time domain analysis is plotted in Fig. 5-8 and the inertia force plotted in Fig. 5-9. The results from regular wave response analyses are also plotted for comparison. The plots indicate that the inertia force from spectral wave response match the time domain results quite closely, within 20%, except for 15 second period. For this period, the inertia force itself is relatively small. Time domain results have some waviness, this is probably because of averaging with limited number of seeds. The regular wave response shows a dip near 13 sec and increases thereafter. The dip is because of deamplification. The increase in response beyond this is probably due to wave reinforcement.

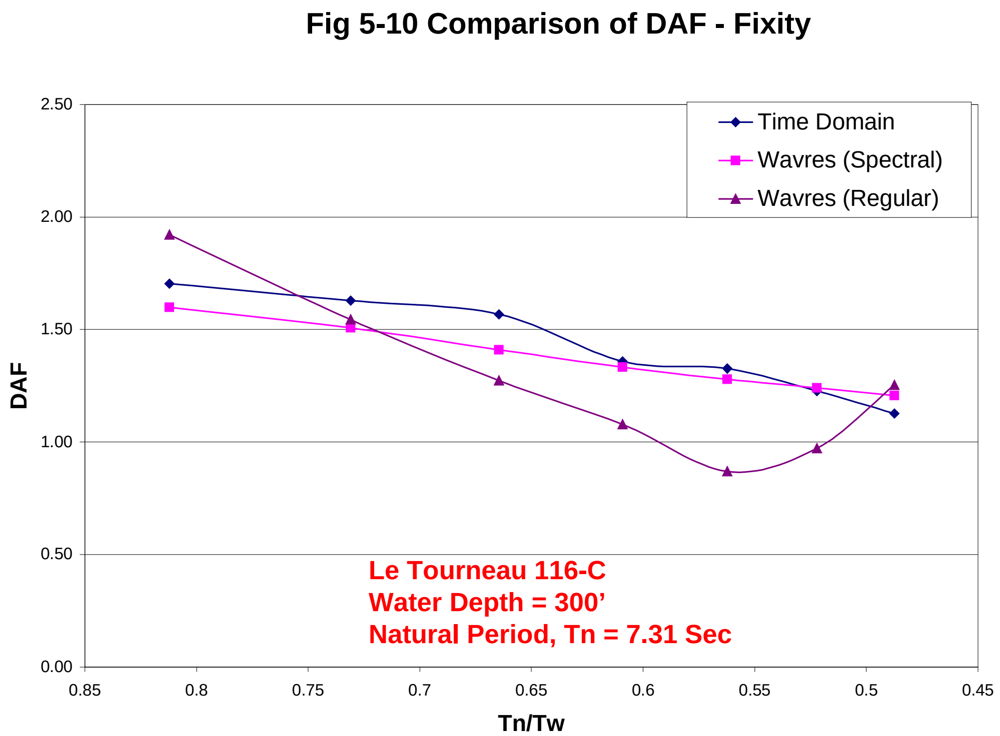
The DAF is shown in Table 5-8 and plotted in Fig. 5-10. Again the spectral wave response results show excellent comparison with time domain. The DAFs from regular wave analyses are also plotted. The spectral wave response results vary on either side of the time domain DAF generally within –10 to +7% of the time domain results.

Table 5-8 Comparison of DAF

Tp Sec	Tn/Tw	Regular Wave Response	Random Time Domain	Spectral Wave Response	
			DAF	DAF	% Diff.
9	0.81	1.92	1.70	1.60	-5.88%
10	0.73	1.54	1.63	1.51	-10.19%
11	0.66	1.27	1.57	1.41	-10.19%
12	0.60	1.08	1.36	1.33	-2.21%
13	0.56	0.87	1.33	1.28	-3.76%
14	0.52	0.97	1.23	1.24	0.81%
15	0.49	1.25	1.13	1.21	7.08%

Fig 5-9 Comparison of Inertia Force - Fixity





The design force comprising of regular static and inertia force from spectral calculations are shown in Table 5-9 and plotted in Fig. 5-11. This shows that the spectral wave response follows a smooth trend over the period ranges similar to the random time domain, the forces are within –5 to +9% of each other.

Table 5-9 Comparison of Design Force

Tp Sec	Random Time Domain	Spectral Wave Response	
	Base Shear kip	Base Shear kip	% Diff.
9	799	813	1.75%
10	833	818	-1.80%
11	863	817	-5.33%
12	819	825	0.73%
13	841	841	0.00%
14	832	862	3.61%
15	812	882	8.62%

The corresponding design overturning moments at the base of the spudcan are tabulated in Table 5-10 and plotted in Fig. 5-12.

Fig 5-11 Comparison of Design Force

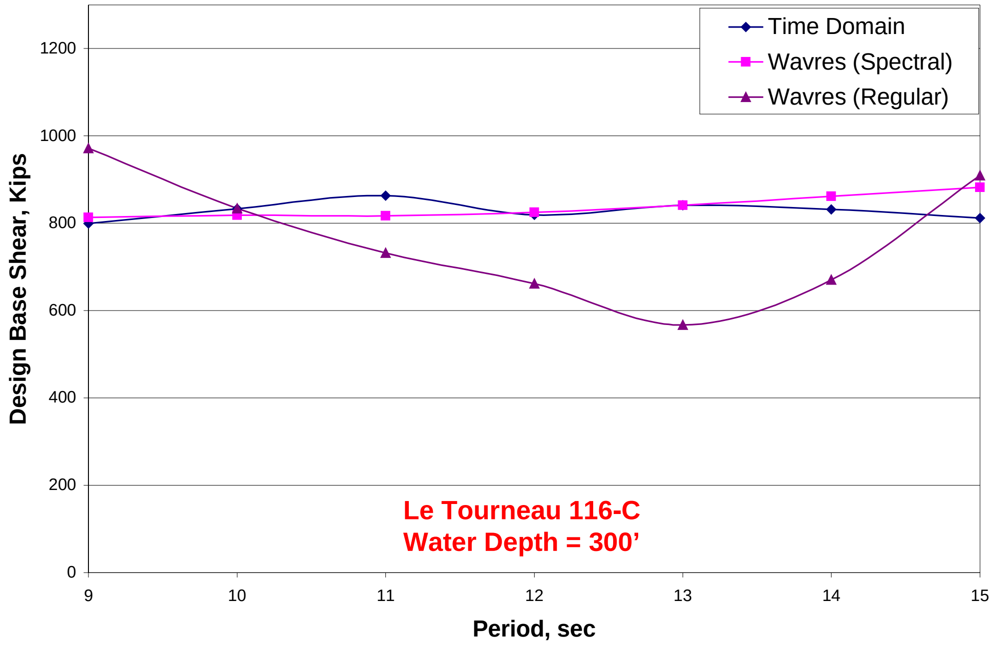
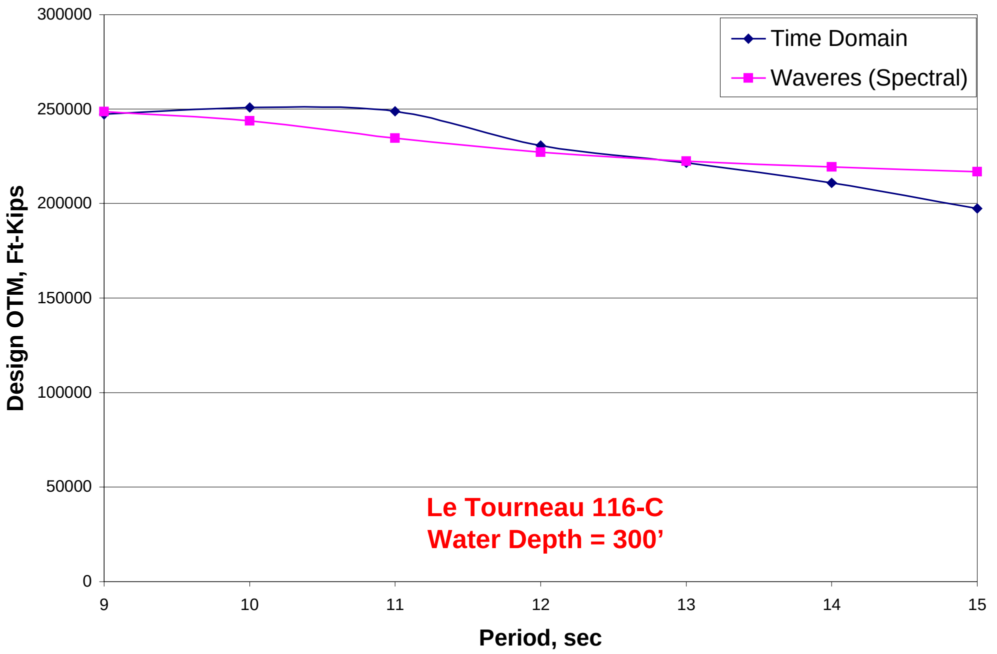


Table 5-10 Comparison of Design OTM

Tp Sec	Random Time Domain	Spectral Wave Response	
	OTM kip-ft	OTM kip-ft	% Diff.
9	247216	248747	0.62%
10	250795	243773	-2.80%
11	248817	234642	-5.70%
12	230652	227197	-1.50%
13	221543	222476	0.42%
14	210943	219314	3.97%
15	197395	216961	9.91%

The design OTM using spectral wave response is within –3 to +10% of the time domain results.

Fig 5-12 Comparison of Design OTM - Fixity



6 LETOURNEAU'S 116-C MODEL – 250 FT WATER DEPTH

The particulars of the 3-leg jack-up model for 250-ft water depth are similar to the 300-ft depth model and shown in Table 6-1.

Table 6-1 Details of 116-C Jack-up Rig – 250' Depth

Leg length	477 ft
Air Gap	50 ft
Spud can penetration	25 ft
Support Location	10ft below mudline
Elevated weight	15,100 kips
Leg weight	2683 kips each
Spud can weight	362 kips each
Natural Period - Pinned	9.22 sec
Natural Period – Fixity	6.41 sec
Equivalent Diameter	87.29 in
Drag Coefficient, C_D	4.92
Inertia Coefficient, C_M	2.0

6.1 RESPONSE FOR REGULAR WAVES

The responses to regular waves using wave response and SDOF have already been compared in previous section. So this is not repeated here. Wherever pertinent, response to regular waves have been included in the discussions that follow for irregular waves.

6.2 RESPONSE FOR IRREGULAR WAVES

6.2.1 Pinned Case

Using the transfer function for base shear, the response in irregular waves for a P-M spectrum with H_s=20.69 ft has been evaluated for a range of peak wave periods. The methodology for evaluation of the response by spectral wave response and random time domain has already been described in the previous section.

A comparison of the static wave force is made in Table 6-2. The results are also plotted in Fig. 6-1. Results of static wave force obtained from equivalent regular waves using Dean's Stream Function theory have also been included for comparison. The results indicate that maximum static wave force from spectral wave response based on Drag-Inertia method closely match those from a regular static analysis, wave response values being slightly higher (1-3%), except for wave period 8 sec, where it is about 16%. The regular wave base shear is relatively smaller at this period possibly because of cancellation effect. The *K* factor (Eq. 7) ranges from 0.52-0.68. The random time domain analysis underestimates wave forces by as much as 12-20% from the regular static for almost the entire period range.

Fig 6-1 Comparison of Static Base Shear

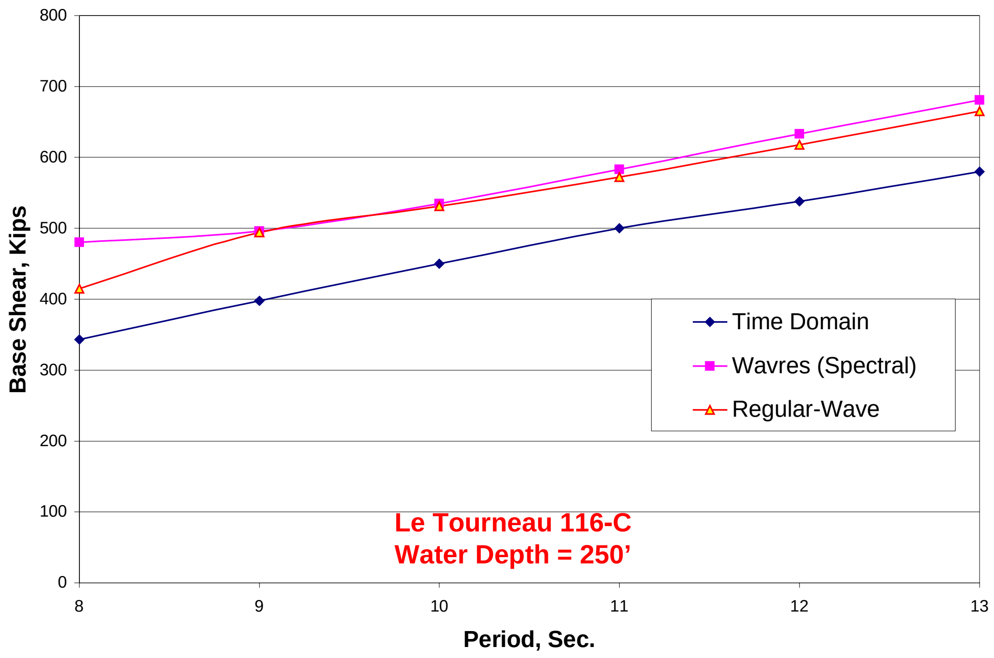


Table 6-2 Comparison of Static Base Shear

Tp Sec	Regular Static Base Shear kips	Random Time Domain		Spectral Wave Response	
		Base Shear kips	% Diff.	Base Shear kips	% Diff.
8	415	343	-17.35%	480	15.66%
9	494	398	-19.43%	496	0.40%
10	531	450	-15.25%	535	0.75%
11	572	500	-12.59%	583	1.92%
12	618	538	-12.94%	633	2.43%
13	665	580	-12.78%	681	2.41%

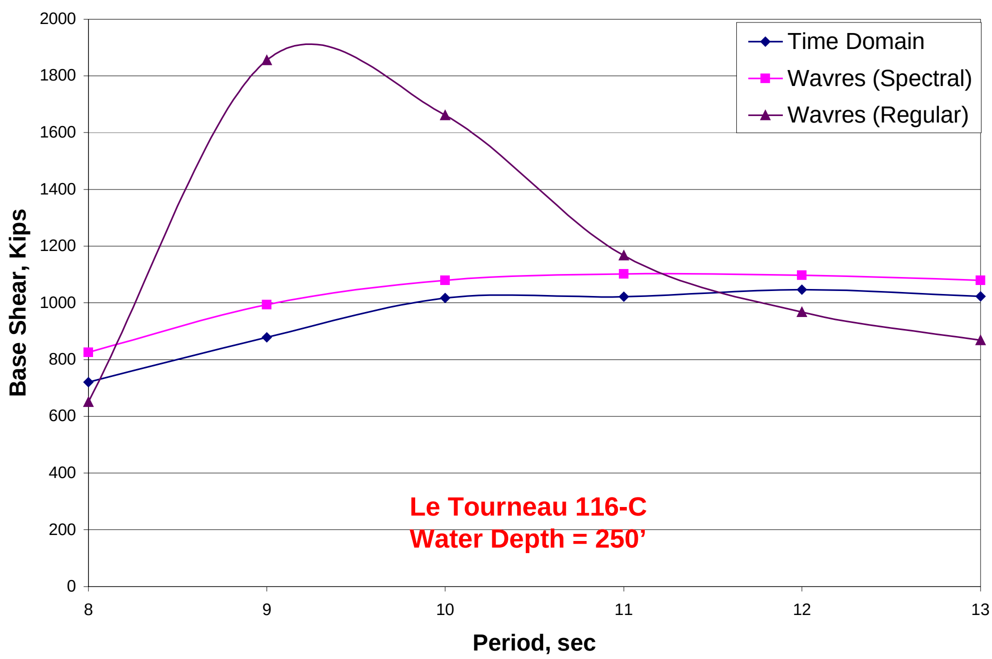
Most probable maximum dynamic response was also computed by spectral wave response and time domain analysis. The maximum inertia force, which is defined as the difference between the dynamic and the static mpme values, is shown in Table 6-3.

Table 6-3 Comparison of Inertia Force

Tp Sec	Regular Wave Response Kips	Random Time Domain		Spectral Wave Response		
		Base Shear mpme, kips	Inertia Force kips	Base Shear mpme, kips	Inertia Force kips	Inertia Force % Diff.
8	651	721	378	826	346	-8.47%
9	1855	878	480	994	498	3.75%
10	1661	1017	567	1080	545	-3.88%
11	1167	1022	522	1102	519	-0.57%
12	967	1046	508	1097	464	-8.66%
13	868	1023	443	1080	399	-9.93%

The total dynamic base shear from the wave response and random time domain analysis is plotted in Fig. 6-2 and the inertia force plotted in Fig. 6-3. The results from regular wave response analyses are also plotted for comparison. The plots indicate that the inertia force from spectral wave response match the time domain results quite closely, within –10 to +3%. The regular wave response shows the traditional peak near the natural period and overestimates the response near it. For wave periods about 15-23% (12 sec <T_p<8 sec,) away from the natural period, all the regular wave analyses underestimate the dynamic response. This is due to the fact that for regular waves, all energy is still contained at the wave period, which is away from the natural period. Whereas, for a spectral simulation, the tail of the wave spectrum could still overlap the natural period, which increases the dynamic response for the time-domain irregular wave analysis.

Fig 6-2 Comparison of Total Base Shear - Pinned



The DAF computed using Eq. 9 is shown in Table 6-4 and plotted in Fig. 6-4. Again the spectral wave response results show excellent comparison with time domain. The DAFs from regular wave analyses are also plotted. The spectral wave response results are within 7-18% of the time domain results.

Table 6-4 Comparison of DAF

Tp Sec	Tn/Tw	Regular Wave Response	Random Time Domain	Spectral Wave Response	
			DAF	DAF	% Diff.
8	1.15	1.57	2.10	1.72	-18.10%
9	1.02	3.76	2.21	2.00	-10.62%
10	0.92	3.13	2.26	2.02	-10.62%
11	0.84	2.04	2.04	1.89	-7.35%
12	0.77	1.56	1.94	1.73	-10.82%
13	0.71	1.31	1.76	1.59	-9.66%

Fig 6-3 Comparison of Inertia Force - Pinned

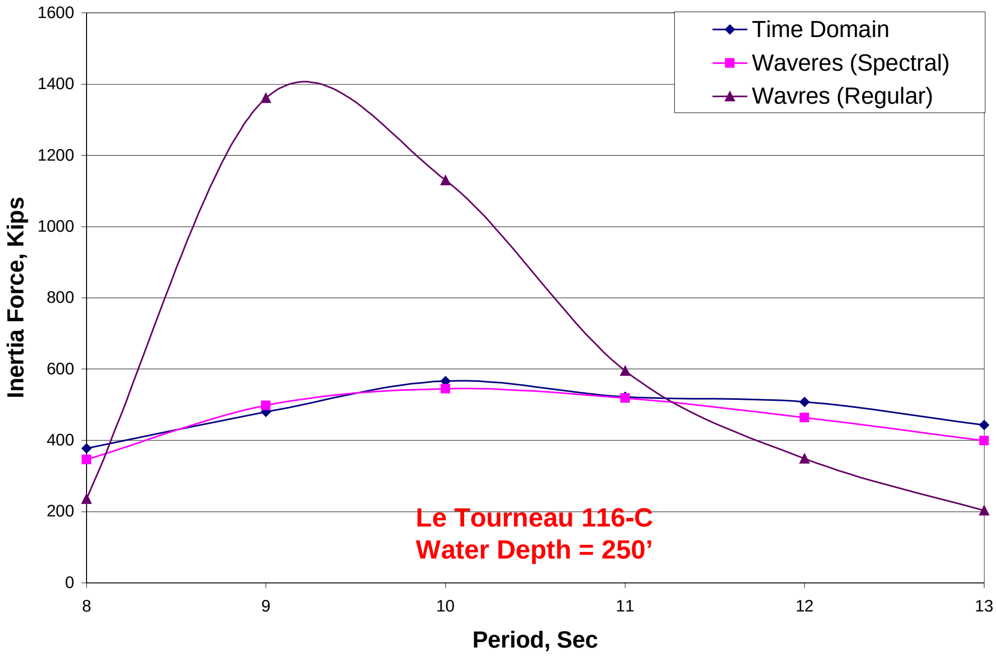
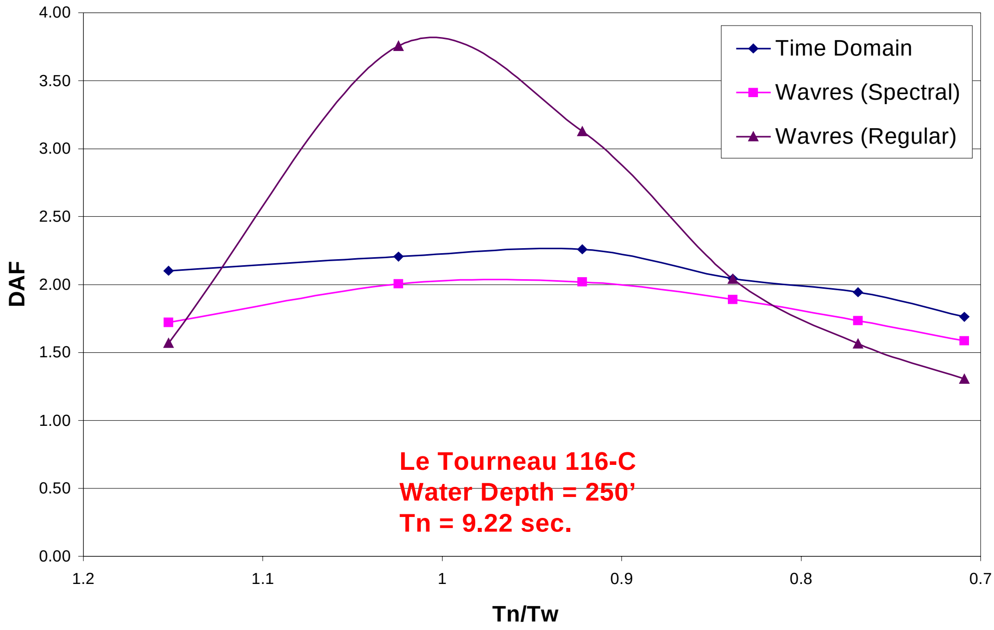


Fig 6-4 Comparison of DAF - Pinned



The design force comprising of regular static and inertia force from spectral calculations are shown in Table 6-5 and plotted in Fig. 6-5. This shows that the spectral wave response follows a smooth trend over the period ranges similar to the random time domain, within about –4 to +2%.

Fig 6-5 Comparison of Design Force - Pinned

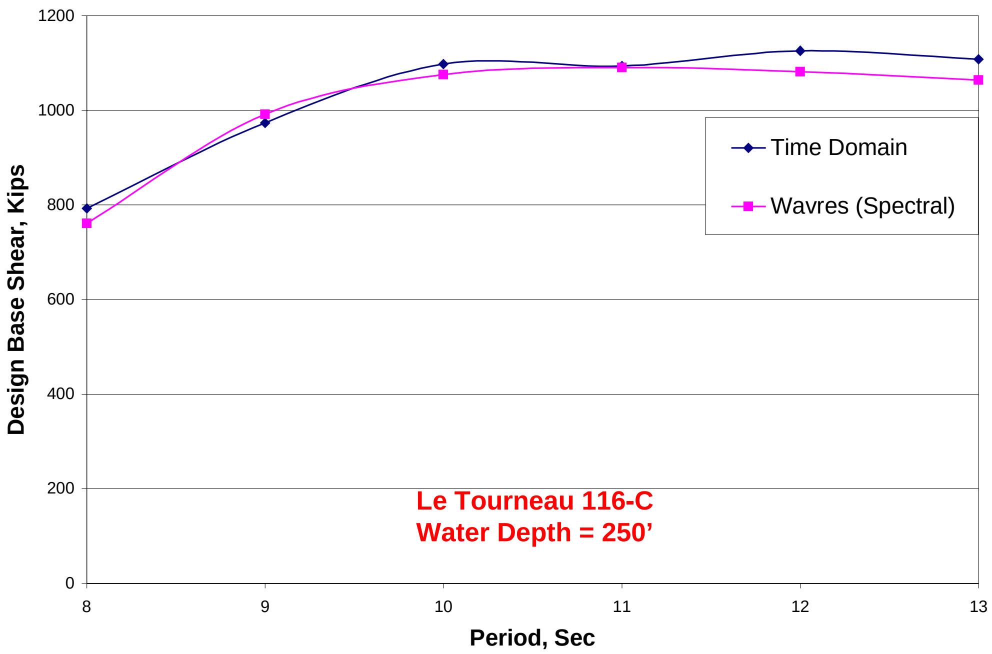


Table 6-5 Comparison of Design Force

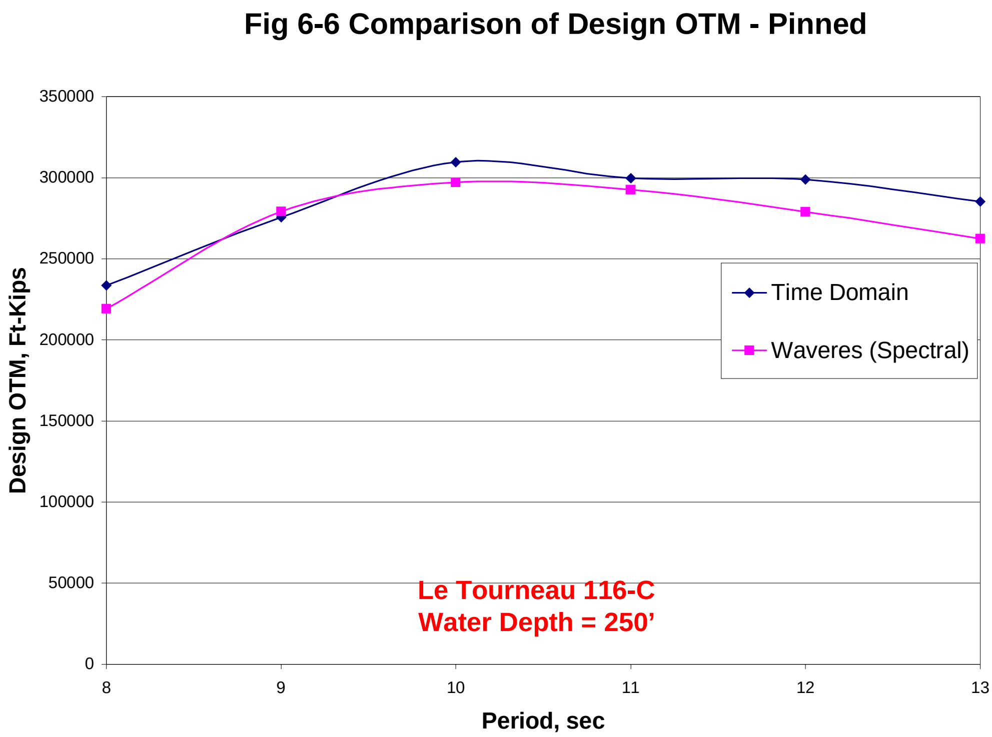
Tp Sec	Random Time Domain	Spectral Wave Response	
	Base Shear kip	Base Shear kip	% Diff.
8	793	761	-4.04%
9	974	992	1.85%
10	1098	1076	-2.00%
11	1094	1091	-0.27%
12	1126	1082	-3.91%
13	1108	1064	-3.97%

The corresponding design overturning moments at the base of the spudcan are tabulated in Table 6-6 and plotted in Fig. 6-6.

Table 6-6 Comparison of Design OTM

Tp Sec	Random Time Domain	Spectral Wave Response	
	OTM kip-ft	OTM kip-ft	% Diff.
8	233515	219077	-6.18%
9	275583	279222	1.32%
10	309595	297143	-4.02%
11	299777	292613	-2.39%
12	298815	279034	-6.62%
13	285271	262354	-8.03%

The design OTM using spectral wave response is within –8 to +1 % of the time domain results.



6.2.2 Fixity Case

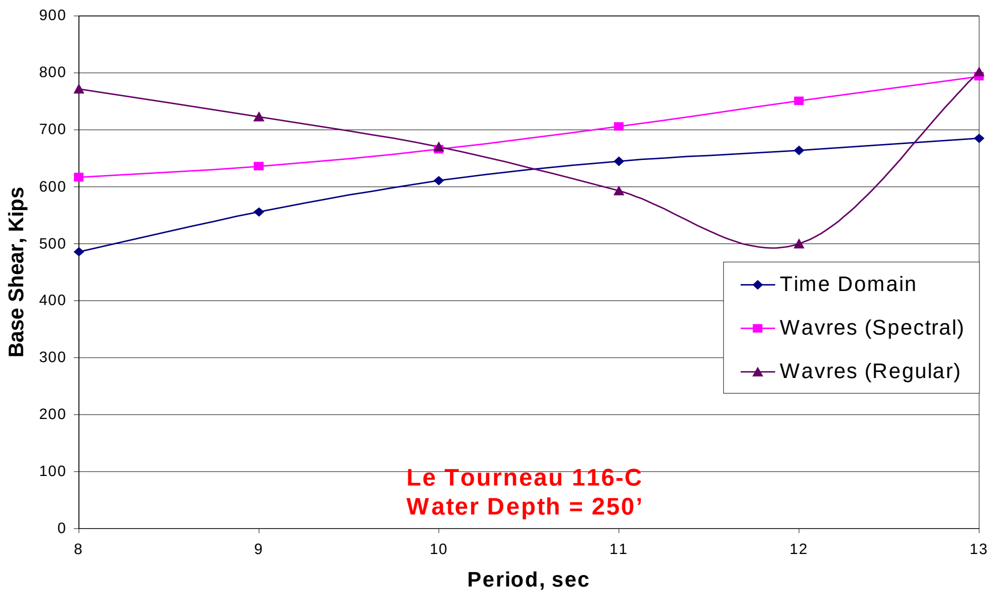
Similar analyses have been performed for spud cans with fixity. The static wave force is the same for the pinned condition. Most probable maximum dynamic response was computed by spectral wave response and time domain analysis. The maximum inertia force is shown in Table 6-7.

Table 6-7 Comparison of Inertia Force

Tp Sec	Regular Wave Response kips	Random Time Domain		Spectral Wave Response		
		Base Shear mpme, kips	Inertia Force kips	Base Shear mpme, kips	Inertia Force kips	Inertia Force % Diff.
8	772	486	143	617	137	-4.20%
9	723	556	158	636	140	-11.39%
10	670	611	161	666	131	-18.63%
11	593	645	145	706	123	-15.17%
12	500	664	126	751	118	-6.35%
13	802	685	105	794	113	7.62%

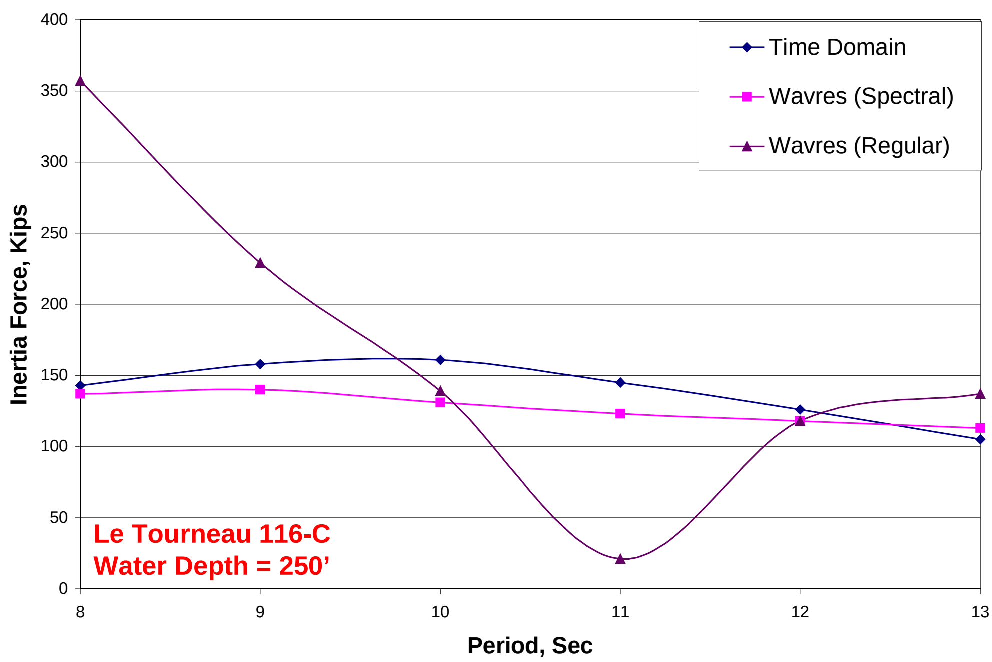
The total dynamic base shear from the wave response and random time domain analysis is plotted in Fig. 6-7 and the inertia force plotted in Fig. 6-8. The results from regular wave response analyses are also plotted for comparison. The plots indicate that the inertia force from spectral wave response match the time domain results quite closely, within –19 to +7%. The regular wave response shows a dip near 11 sec and increases thereafter. The dip is because of deamplification. The increase in response beyond this period is probably due to longer wavelength.

Fig 6-7 Comparison of Total Base Shear - Fixity



The DAF is shown in Table 6-8 and plotted in Fig. 6-9. Again the spectral wave response results show excellent comparison with time domain. The DAFs from regular wave analyses

Fig 6-8 Comparison of Inertia Force - Fixity

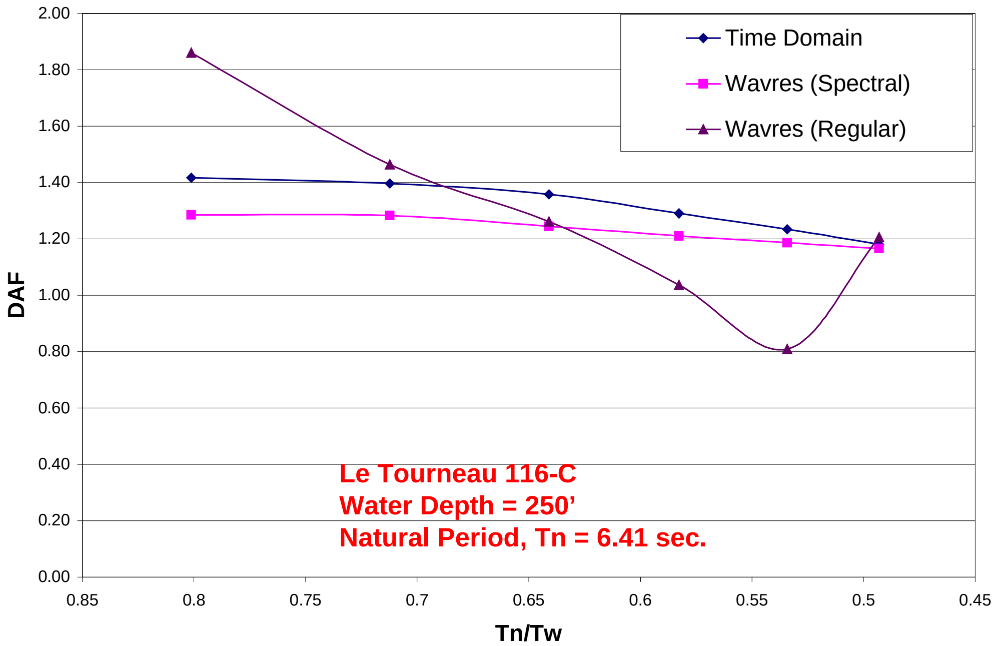


are also plotted. The spectral wave response results are within 0-10% of the time domain results.

Table 6-8 Comparison of DAF

Tp Sec	Tn/Tw	Regular Wave Response	Random Time Domain	Spectral Wave Response	
			DAF	DAF	% Diff.
8	0.80	1.86	1.42	1.29	-9.15%
9	0.71	1.46	1.40	1.28	-8.82%
10	0.64	1.26	1.36	1.24	-8.82%
11	0.58	1.04	1.29	1.21	-6.20%
12	0.53	0.81	1.23	1.19	-3.25%
13	0.49	1.21	1.18	1.17	-0.85%

Fig 6-9 Comparison of DAF - Fixity



The design force comprising of regular static and inertia force from spectral calculations are shown in Table 6-9 and plotted in Fig. 6-10. This shows that the spectral wave response follows a smooth trend over the period ranges similar to the random time domain, the forces are within –4 to +1% of each other.

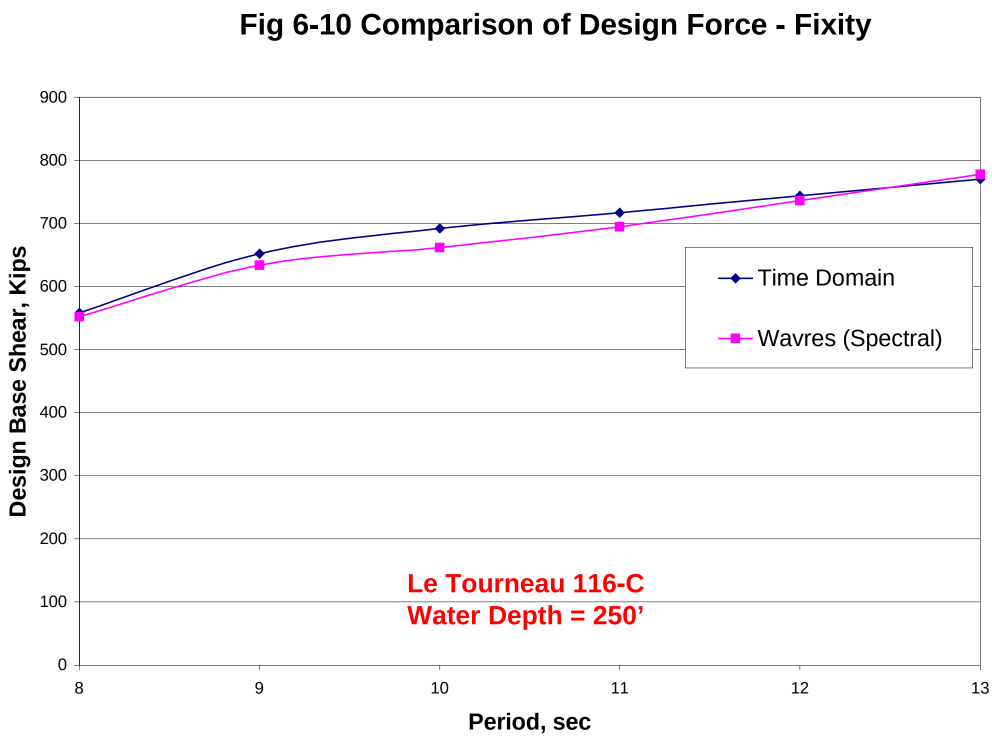


Table 6-9 Comparison of Design Force

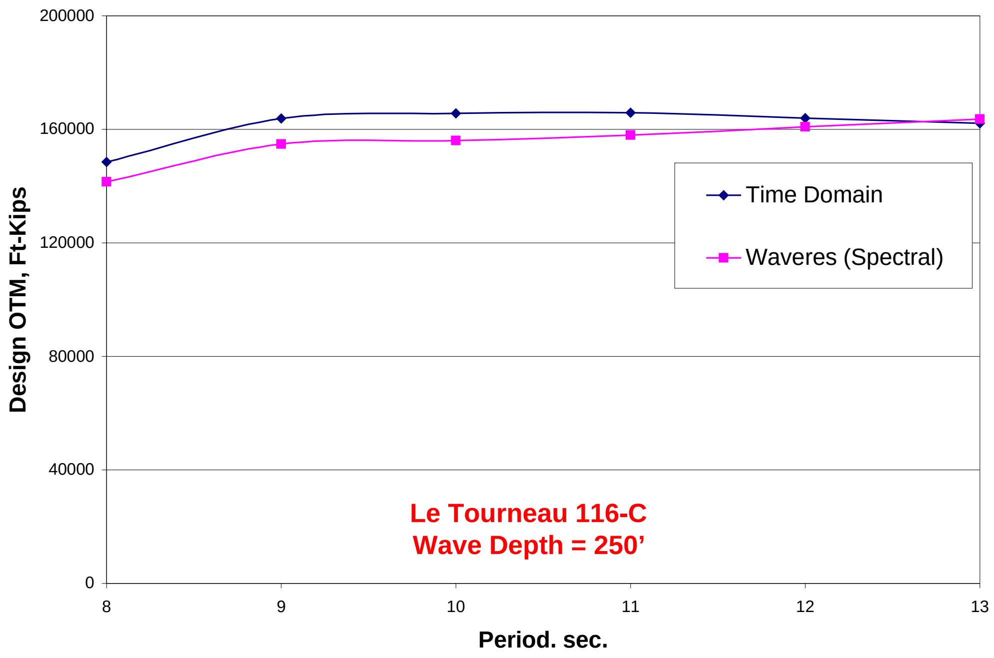
Tp Sec	Random Time Domain	Spectral Wave Response	
	Base Shear kip	Base Shear kip	% Diff.
8	558	552	-1.08%
9	652	634	-2.76%
10	692	662	-4.34%
11	717	695	-3.07%
12	744	736	-1.08%
13	770	778	1.04%

The corresponding design overturning moments at the base of the spudcan are tabulated in Table 6-10 and plotted in Fig. 6-11.

Table 6-10 Comparison of Design OTM

Tp Sec	Random Time Domain	Spectral Wave Response	
	OTM kip-ft	OTM kip-ft	% Diff.
8	148490	141497	-4.71%
9	163764	154827	-5.46%
10	165618	156044	-5.78%
11	165911	157996	-4.77%
12	163921	160952	-1.81%
13	162166	163577	0.87%

Fig 6-11 Comparison of Design OTM - Fixity



The design OTM using spectral wave response is within 0-6% of the time domain results.

7 FRIEDE & GOLDMAN'S L-780 MODEL – 300 FT WATER DEPTH

The particulars of the 3-leg jack-up model for 300 ft water depth shown in Table 7-1.

Table 7-1 Details of L-780 Jack-up Rig – 300' Depth

Leg length	417 ft
Air Gap	37 ft
Spud can penetration	17 ft
Support Location	10ft below mudline
Elevated weight	12,500 kips
Leg weight	1186 kips each
Spud can weight	350 kips each
Natural Period – Pinned	9.70 sec
Natural Period – Fixity	8.39 sec
Equivalent Diameter	48.05 in
Drag Coefficient, C_D	2.98
Inertia Coefficient, C_M	2.0

7.1 RESPONSE FOR REGULAR WAVES

The responses to regular waves using wave response and SDOF have already been compared for 116-C model. So this is not repeated here. Wherever pertinent response to regular waves have been included in the discussions that follow for irregular waves.

7.2 RESPONSE FOR IRREGULAR WAVES

7.2.1 Pinned Case

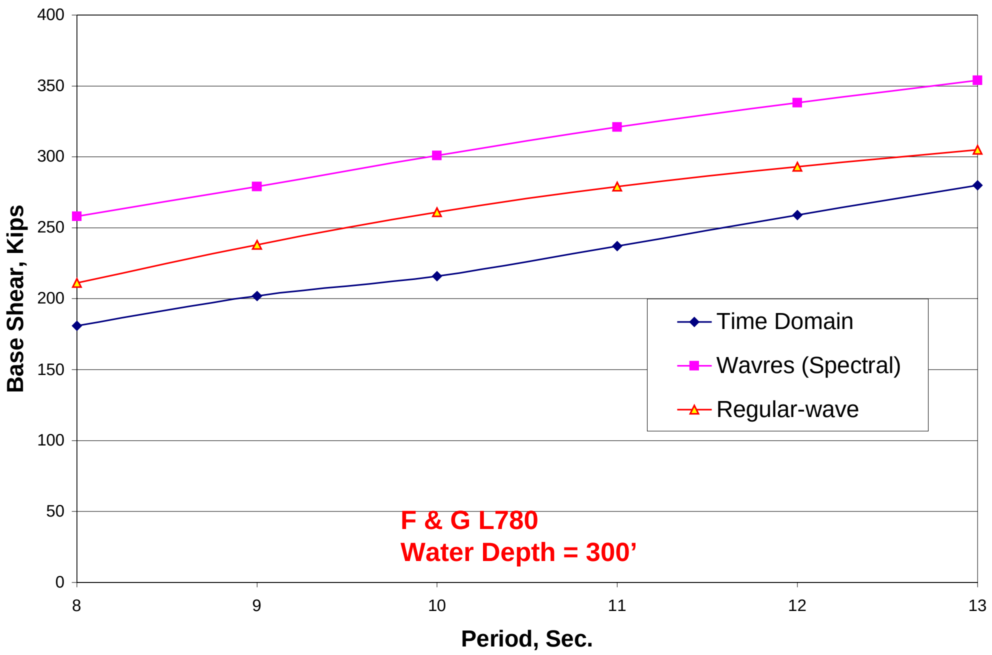
Using the transfer function for base shear, the response in irregular waves for a P-M spectrum with H_s=18.82 ft has been evaluated for a range of peak wave periods. A current of 1.5 knots has been included in the wave model. The methodology for evaluation of the response by spectral wave response and random time domain has already been described in the previous section.

A comparison of the static wave force is made in Table 7-2. The results are also plotted in Fig. 7-1. Results of static wave force obtained from equivalent regular waves using Dean's Stream Function theory have also been included for comparison. The results indicate that maximum static wave force from spectral wave response based on Drag-Inertia method closely match those from a regular static analysis, wave response values being 15-22% higher. The *K* factor (Eq. 7) ranges from 0.96-1.09 which is considerable higher than for 116-C models. This is mostly because of higher current used. The random time domain analysis underestimates wave forces by as much as 8-17% from the regular static for almost the entire period range. It appears that Drag-Inertia method tends to overestimate the static wave force maxima when *K* factors are large.

Table 7-2 Comparison of Static Base Shear

Tp Sec	Regular Static Base Shear Kips	Random Time Domain		Spectral Wave Response	
		Base Shear kips	% Diff.	Base Shear kips	% Diff.
8	211	181	-14.22%	258	22.27%
9	238	202	-15.13%	279	17.23%
10	261	216	-17.24%	301	15.33%
11	279	237	-15.05%	321	15.05%
12	293	259	-11.60%	338	15.36%
13	305	280	-8.20%	354	16.07%

Fig 7-1 Comparison of Static Base Shear

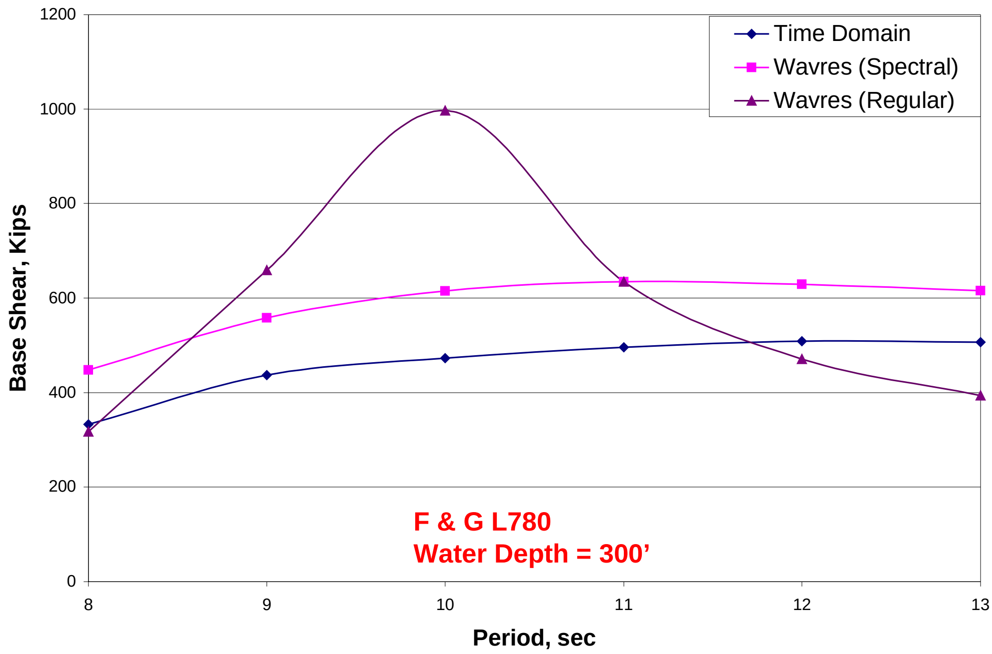


Most probable maximum dynamic response was also computed by spectral wave response and time domain analysis. The maximum inertia force, which is defined as the difference between the dynamic and the static mpme values, is shown in Table 7-3.

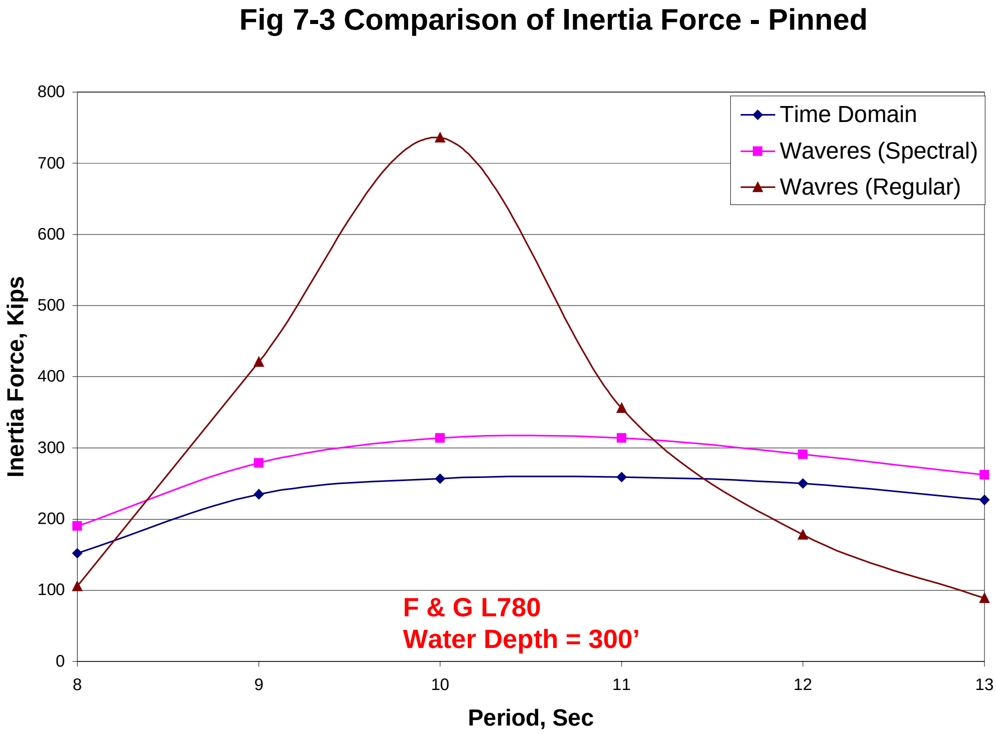
Table 7-3 Comparison of Inertia Force

Tp Sec	Regular Wave Response Kips	Random Time Domain		Spectral Wave Response		
		Base Shear mpme, kips	Inertia Force kips	Base Shear mpme, kips	Inertia Force kips	Inertia Force % Diff.
8	317	333	152	448	190	25.00%
9	659	437	235	558	279	18.72%
10	997	473	257	615	314	22.18%
11	635	496	259	635	314	21.24%
12	471	509	250	629	291	16.40%
13	394	507	227	616	262	15.42%

Fig 7-2 Comparison of Total Base Shear - Pinned



The total dynamic base shear from the wave response and random time domain analysis is plotted in Fig. 7-2 and the inertia force plotted in Fig. 7-3. The results from regular wave response analyses are also plotted for comparison. The plots indicate that the inertia force from spectral wave response match the time domain results quite closely, within 15-25%. The regular wave response shows the traditional peak near the natural period and overestimates the response near it. For wave periods beyond about 20% (11.5 sec $<T_p<$8.2 sec,) from the natural period, all the regular wave analyses underestimate the dynamic response. This is due to the fact that for regular waves, all energy is still contained at the wave period, which is away from the natural period, whereas, for a spectral simulation, the tail of the wave spectrum could still overlap the natural period. This increases the dynamic response for the time-domain and the wave response irregular wave analyses.

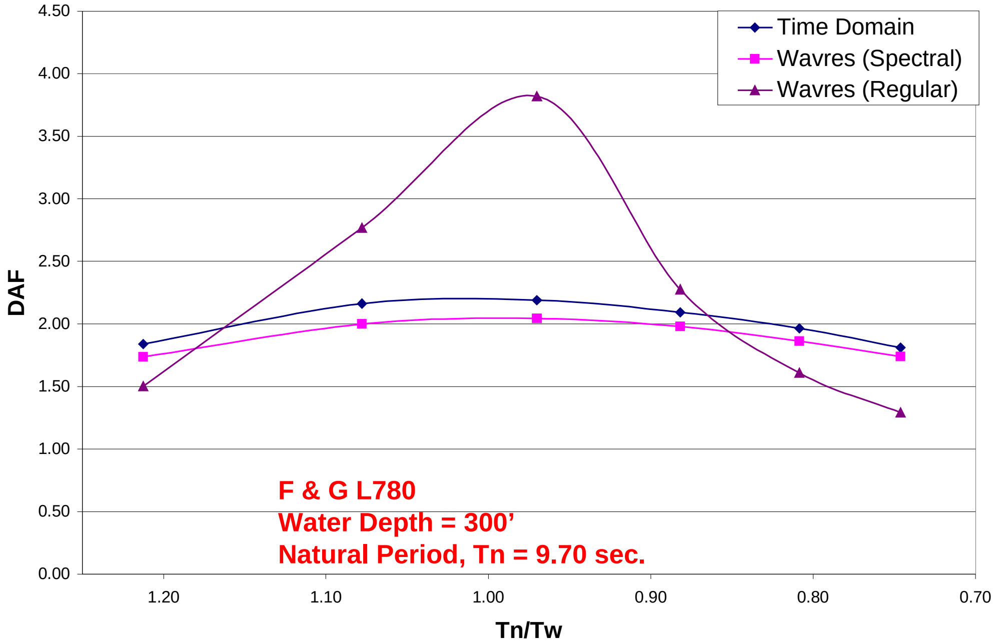


The DAF computed using Eq. 9 is shown in Table 7-4 and plotted in Fig. 7-4. Again the spectral wave response results show excellent comparison with time domain. The DAFs from regular wave analyses are also plotted. The spectral wave response results are within 3-7% of the time domain results.

Table 7-4 Comparison of DAF

Tp Sec	Tn/Tw	Regular Wave Response	Random Time Domain	Spectral Wave Response	
			DAF	DAF	% Diff.
8	1.21	1.50	1.84	1.74	-5.43%
9	1.08	2.77	2.16	2.00	-6.85%
10	0.97	3.82	2.19	2.04	-6.85%
11	0.88	2.28	2.09	1.98	-5.26%
12	0.81	1.61	1.97	1.86	-5.58%
13	0.75	1.29	1.81	1.74	-3.87%

Fig 7-4 Comparison of DAF - Pinned



The design force comprising of regular static and inertia force from spectral calculations are shown in Table 7-5 and plotted in Fig. 7-5. This shows that the spectral wave response follows a smooth trend over the period ranges similar to the random time domain, only the forces are about 6-11% higher.

Table 7-5 Comparison of Design Force

Tp Sec	Random Time Domain	Spectral Wave Response	
	Base Shear kip	Base Shear kip	% Diff.
8	363	401	10.47%
9	473	517	9.30%
10	518	575	11.00%
11	538	593	10.22%
12	543	584	7.55%
13	532	567	6.58%

The corresponding design overturning moments at the base of the spudcan are tabulated in Table 7-6 and plotted in Fig. 7-6.

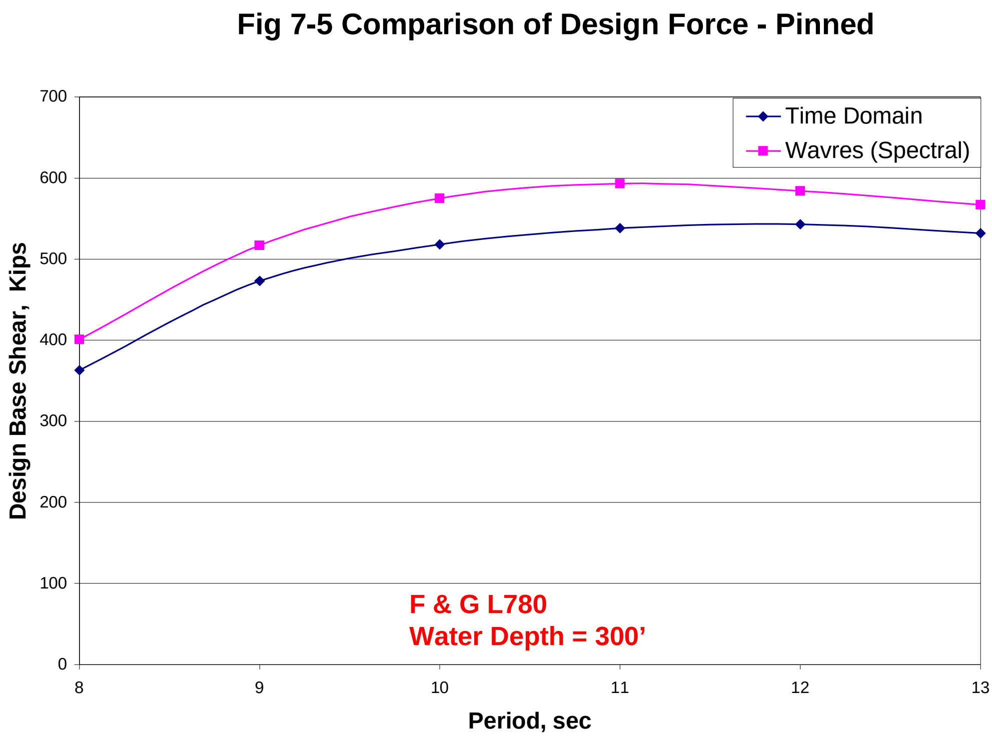


Table 7-6 Comparison of Design OTM

Tp Sec	Random Time Domain	Spectral Wave Response	
	OTM kip-ft	OTM kip-ft	% Diff.
8	117740	126087	7.09%
9	151917	162068	6.68%
10	164145	178352	8.66%
11	166996	181086	8.44%
12	166110	175019	5.36%
13	157430	166283	5.62%

The design OTM using spectral wave response is within 5-9% of the time domain results.

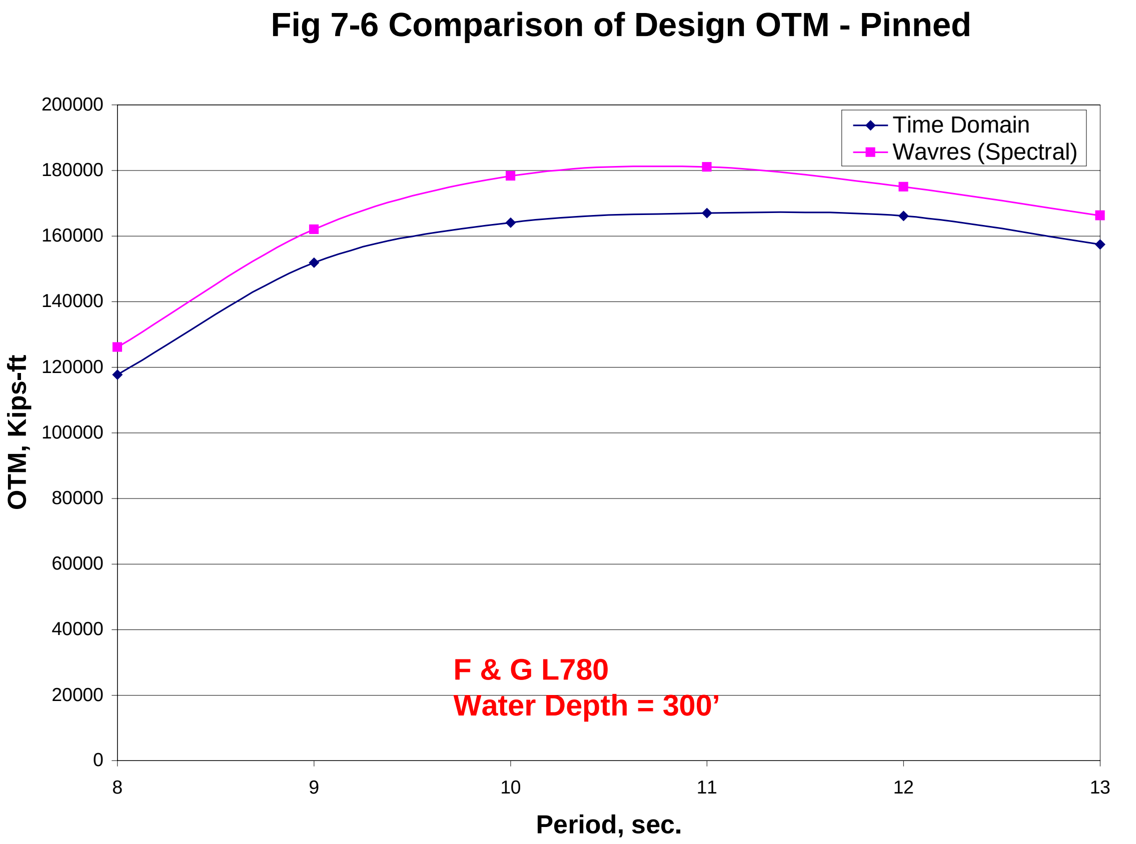


Fig 7-6 Comparison of Design OTM - Pinned
Time Domain
Wavres (Spectral)
OTM, Kips-ft
200000
180000
160000
140000
120000
100000
80000
60000
40000
20000
0
8
9
10
11
12
13
Period, sec.
F & G L780
Water Depth = 300’

7.2.2 Fixity Case

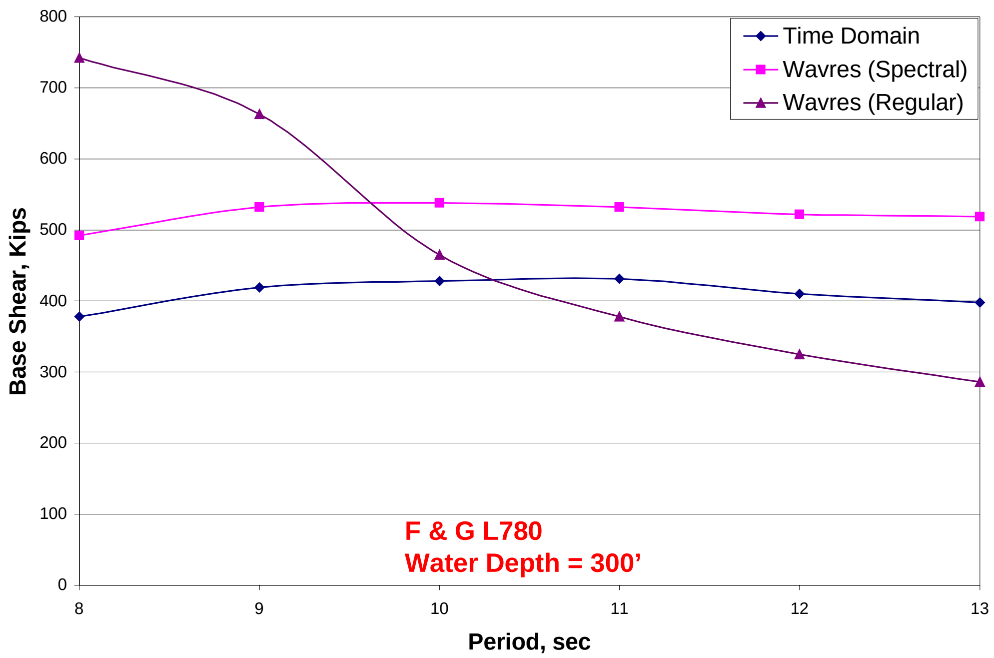
Similar analyses have been performed for spud cans with fixity. The static wave force is the same for the pinned condition. Most probable maximum dynamic response was computed by spectral wave response and time domain analysis. The maximum inertia force is shown in Table 7-7.

Table 7-7 Comparison of Inertia Force

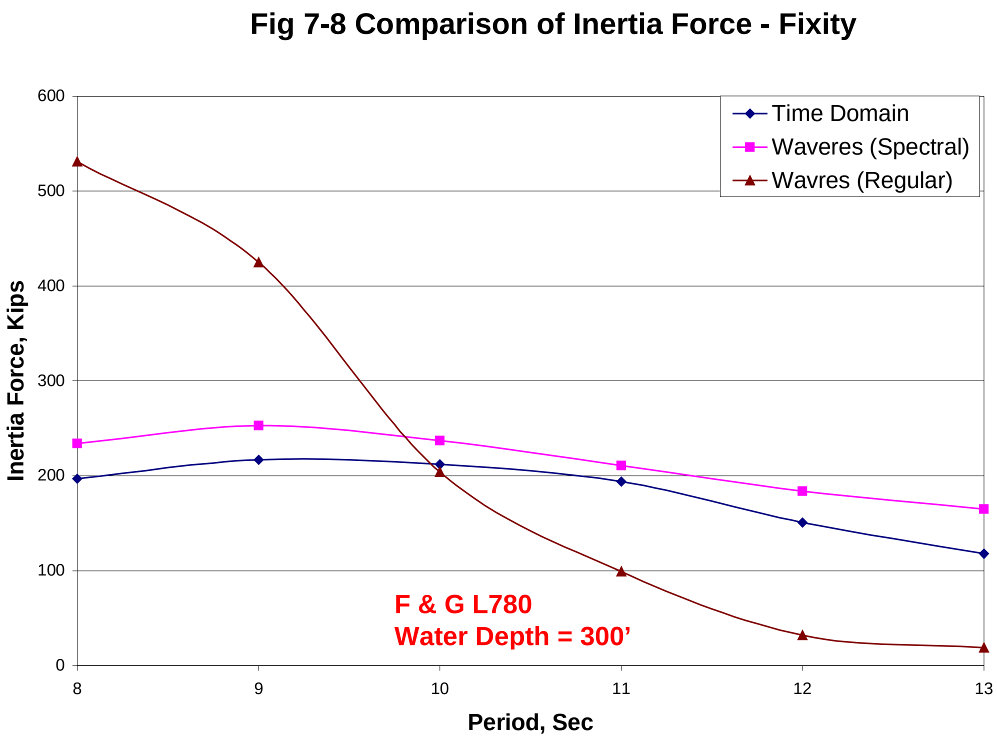
Tp Sec	Regular Wave Response Kips	Random Time Domain		Spectral Wave Response		
		Base Shear mpme, kips	Inertia Force kips	Base Shear mpme, kips	Inertia Force kips	Inertia Force % Diff.
8	742	378	197	492	234	18.78%
9	663	419	217	532	253	16.59%
10	465	428	212	538	237	11.79%
11	378	431	194	532	211	8.76%
12	325	410	151	522	184	21.85%
13	286	398	118	519	165	39.83%

The total dynamic base shear from the wave response and random time domain analysis is plotted in Fig. 7-7 and the inertia force plotted in Fig. 7-8. The results from regular wave response analyses are also plotted for comparison. The plots indicate that the inertia forces

Fig 7-7 Comparison of Total Base Shear - Fixity



from spectral wave response analyses are within 8-40% of the time domain results. The regular wave response shows a lower inertia force above 10.3 sec period. The difference is mainly because of spectral effect.



The DAF is shown in Table 7-8 and plotted in Fig. 7-9. Again the spectral wave response results show excellent comparison with time domain. The DAFs from regular wave analyses are also plotted. The spectral wave response results vary on either side of the time domain DAF generally within –10 to +4% of the time domain results.

Table 7-8 Comparison of DAF

Tp Sec	Tn/Tw	Regular Wave Response	Random Time Domain	Spectral Wave Response	
			DAF	DAF	% Diff.
8	1.05	3.52	2.09	1.91	-8.61%
9	0.93	2.79	2.07	1.91	-9.60%
10	0.84	1.78	1.98	1.79	-9.60%
11	0.76	1.35	1.82	1.66	-8.79%
12	0.70	1.11	1.58	1.54	-2.53%
13	0.65	0.94	1.42	1.47	3.52%

The design force comprising of regular static and inertia force from spectral calculations are shown in Table 7-9 and plotted in Fig. 7-10. This shows that the spectral wave response follows a smooth trend over the period ranges similar to the random time domain, the forces are within 3-11% of each other.

Fig 7-9 Comparison of DAF - Fixity

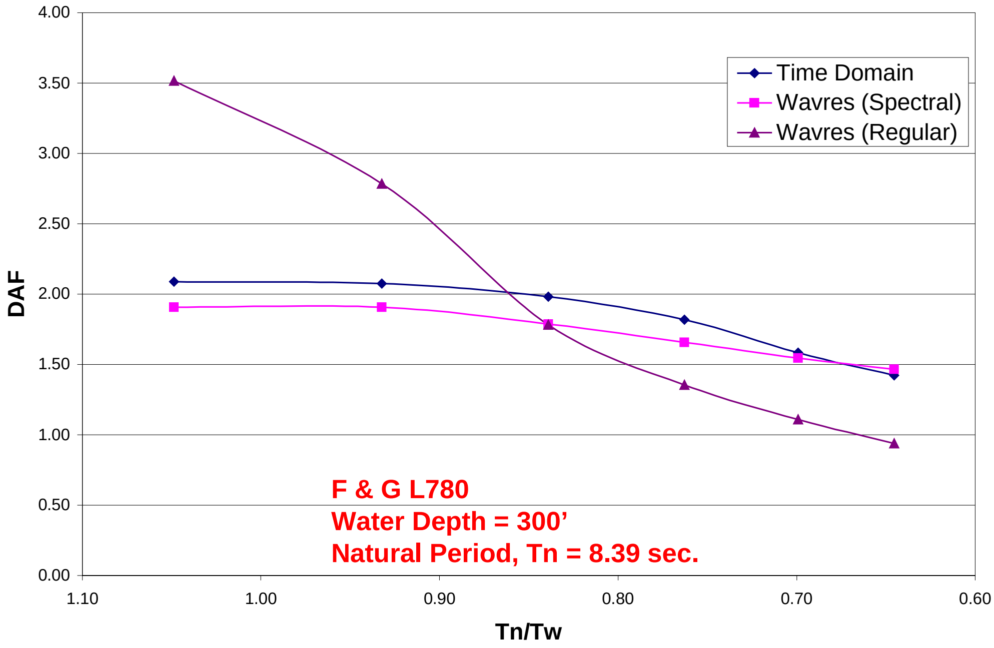


Table 7-9 Comparison of Design Force

Tp Sec	Random Time Domain	Spectral Wave Response	
	Base Shear kip	Base Shear kip	% Diff.
8	408	445	9.07%
9	455	491	7.91%
10	473	498	5.29%
11	473	490	3.59%
12	444	477	7.43%
13	423	470	11.11%

The corresponding design overturning moments at the base of the spudcan are tabulated in Table 7-10 and plotted in Fig. 7-11.

Fig 7-10 Comparison of Design Force - Fixity

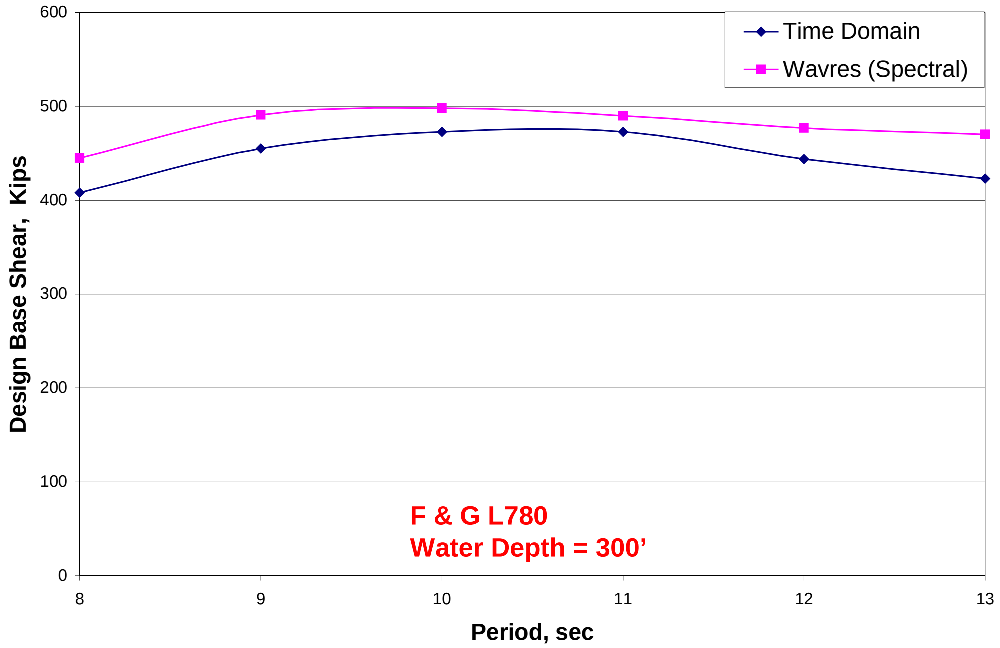
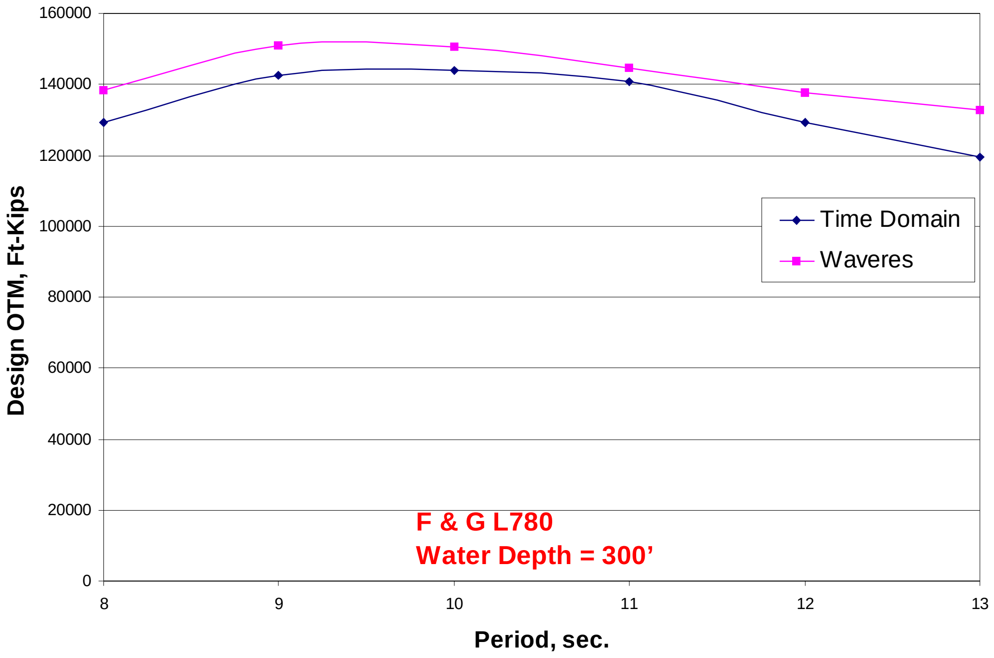


Table 7-10 Comparison of Design OTM

Tp Sec	Random Time Domain	Spectral Wave Response	
	OTM kip-ft	OTM kip-ft	% Diff.
8	129305	138265	6.93%
9	142483	150768	5.81%
10	143918	150516	4.58%
11	140680	144697	2.86%
12	129209	137788	6.64%
13	119454	132590	11.00%

The design OTM using spectral wave response is within 2-11% of the time domain results.

Fig 7-11 Comparison of Design OTM - Fixity



8 SUMMARY & CONCLUSIONS

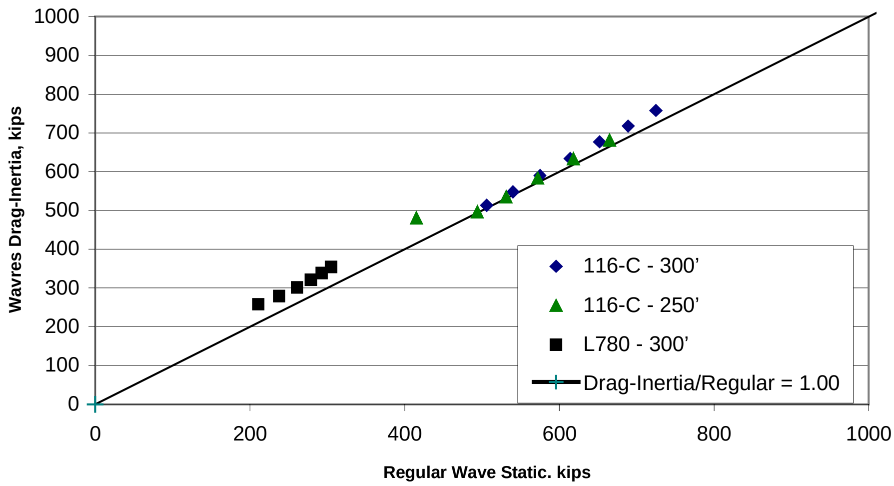
8.1 SUMMARY

A new efficient frequency-domain method has been presented which accounts for some of the hydrodynamic nonlinearities present in the dynamics of an offshore structure, such as a jack-up unit. The method is approximate, but much simpler than the elaborate and time-consuming time domain approach.

Two structural models of LeTourneau's 116-C jack-up unit, one for 300 ft and the other for 250 ft water depth, and one model for Friede & Goldman L-780 for 300 ft water depth, have been analyzed in this report. Both the cases of spudcan as pinned and with fixity have been analyzed. Based on the analysis and the results presented here, the following observations may be made:

1. The presence of higher harmonics for static and dynamic base shear is shown in regular waves.
2. Dynamic analysis for regular waves using Wave Response Method is discussed considering hydrodynamic nonlinearities. The results from this analysis exactly match those from time domain calculations.
3. Recommended wave height as a function of period increases sharply with period. An alternate method for selecting the wave height is proposed for the construction of the transfer function.
4. In irregular waves, a new and efficient hybrid method is proposed in which the drag-inertia parameter is used for the static response while the Rayleigh distribution for the dynamic inertia response maxima.
5. Drag-Inertia method has predicted static wave response quite well, average deviation being 8% from regular static. This is demonstrated in Fig. 8-1. However, it appears that this method tends to overestimate the static wave force maxima when *K* factors are large, e.g. for L-780 model. To investigate this further, additional static wave loads

Fig. 8-1 Comparison of Static Wave Force



for both 116-C and L-780 models were run for a different value of current. For L-780 model the additional case was run for zero current (H_s=18.82 ft, T_p=10 sec), while for 116-C model, the additional case was run for 1.5 knot current (H_s=20.69 ft, T_p=10 sec). The results are summarized below:

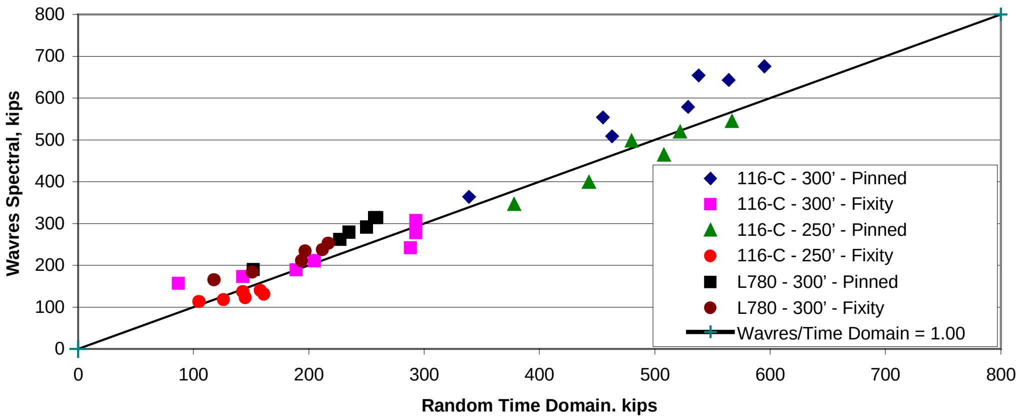
Table 8-1 Comparison of Wave Force with different Current

Method of Analysis	116 C Model		L-780 Model	
	Low Current kip	High Current kip	Low Current kip	High Current kip
Drag-Inertia	548	968	146	301
Regular Wave	540	832	142	261
% difference	1.48%	16.35%	2.82%	15.33%

From the above, it is concluded that the static wave force estimation by drag-inertia method is close to the regular static wave force when current is small, but the difference becomes large for higher current. The difference in estimation is not due to any peculiarity of the models, but inherent to the procedure.

6. It is shown that the proposed method produces consistent results for the dynamic maxima when compared to the time domain results for all the cases considered. This is demonstrated in Fig. 8-2, where the inertia forces from all the cases analyzed has been

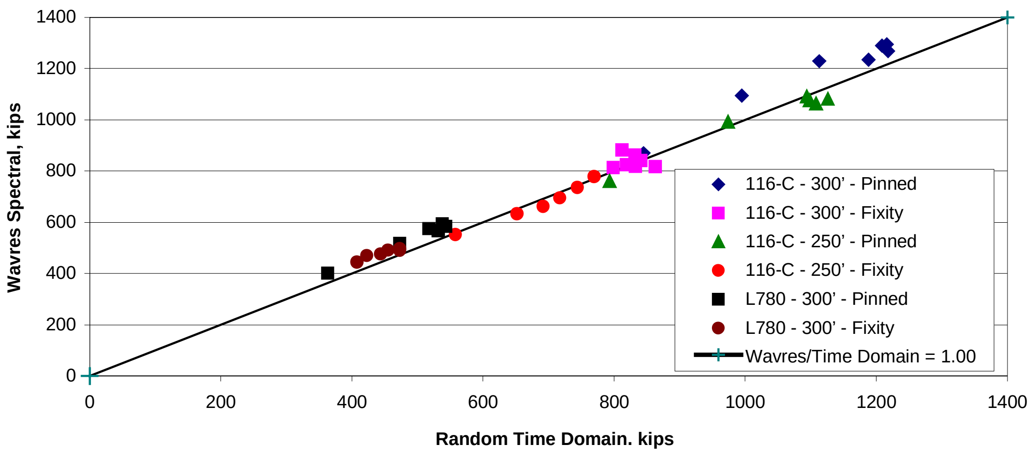
Fig. 8-2 Comparison of Inertia Force



plotted against those from time domain simulation. The average deviation is about 9% from time domain.

7. The design force from spectral wave response analysis, which is defined as the sum of the maximum regular static and the inertia force from Rayleigh extrapolation, match very closely (average deviation 3%) with those from time domain analysis, Fig. 8-3. The same is the case for design overturning moment. For the design forces and moments the static wave forces resulting from Drag-Inertia approach are not utilized.

Fig. 8-3 Comparison of Design Force



8.2 CONCLUSIONS

Wave Response Method is a very powerful tool for dynamic analysis of structures such as jack-up rigs. For regular waves it gives exact results as a time domain analysis. For irregular waves the spectral wave response analysis technique provides a faster and often as accurate estimation of the inertia force as random time domain simulation. The method is efficient for large models and applicable for analysis of linear systems.

9 AREA OF FURTHER RESEARCH

The suitability of the method has been demonstrated for different jack-up models, water depths, wave conditions, and foundation fixity. For high current, the drag-inertia approach appears to lead to higher static wave loads. However, the dynamic response remains unaffected. An area of further research would be to investigate the criteria for foundation linearization. In a nonlinear random time domain analysis, the foundation stiffness can be treated as nonlinear depending on the level of loading on the foundation. In a frequency domain analysis, the foundation stiffness can be included but it has to be independent of loading level. Therefore, the foundation stiffness is linearized in the frequency domain model to make it substantially “equivalent” to the time domain model. No equivalent linear system can model all the aspects of the nonlinear formulation. Here, the objective would be to estimate the inertia force as accurately as possible. In general, several linearization techniques have been proposed in the past, among these the use of “standard deviation spring” appears to be promising (Karunakaran, 1993).

The other area would be inclusion of relative velocity in the formulation. When structural velocities are large compared to water particle velocities, the drag forces should be calculated based on the relative velocity. The relative velocity also leads to higher damping of the system. Further work is needed to include the relative velocity in the formulation.

The method should be applicable for other dynamically sensitive structures such as risers, compliant towers etc. The validity of results should be evaluated for such systems.

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