THE DESIGN OF DEEPWATER JACKETS : DYNAMIC EFFECTS

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1.0 INTRODUCTION

During the design of an offshore jacket structure it is necessary to perform numerous fatigue and extreme load analyses to arrive at an optimum configuration. This iterative process can be performed most simply and cheaply by using a static approach and applying dynamic amplification factors (d.a.f's) to the member stresses derived. This approach is used successfully in the early stages of design of North Sea jackets in 100m-150m of water. This is because the deflected shape of a typical North Sea jacket under storm wave loading approximates closely to the global sway mode of the structure in the appropriate direction.

Extensions of the conventional single degree of freedom (d.a.f.) representation necessary for jackets in deeper waters are described, which take into account the frequency spread of wave energy in a storm sea state and correctly amplify the damage for evaluation of fatigue behaviour.

Two actual jacket designs are used to illustrate and quantify the impact of dynamic effects on the stress levels (see figures 1a and 1b). One was a conventional barge launched design (Jacket 1) for 150m and the other was a self-floater designed for 250m (Jacket 2). The jackets were analysed both dynamically and statically for the same wave climate and the member and hot-spot stresses compared directly. Static back-substitution was used to correct for the higher modes (above mode 30) not explicitly derived.

The environmental conditions assumed are given in Table 1, a northern North Sea wave scatter diagram was assumed for derivation of characteristic waves, figure 2. The waves examined for both structures are given in Table 2. The pile spring stiffnesses for the jacket in 150m of water corresponded to those used for the Eider platform and the 250m jacket assumed battered piles with properties similar to those of the Magnus platform.

The 14 conductors of 30" diameter were explicitly represented in both models and equivalent inertia and drag coefficients were carefully calculated to compensate for the simplified hydrodynamic modelling of the conductor ladder regions. As the conductors are allowed to slide vertically their inertia in the vertical direction is set to zero.

The topside mass of each jacket was taken to be 17,000 Tonnes giving a fundamental sway period for jacket 1 of 3.05 secs and for jacket 2 a fundamental period of 3.47 secs. The two periods were arranged to be similar in order to isolate the effects associated with water depth.

The main aim of this paper is to quantify in realistic circumstances the contribution of frame action and dynamic effects to local stress levels and fatigue damage. The emphasis is on the application of the results of these studies to practical design solutions.

2.0 DYNAMIC AMPLIFICATION FACTORS - GENERAL REMARKS

Because of the high fundamental sway natural period of deep-water jackets it is essential to consider dynamic effects which will result in amplification of the jacket response to time dependent loads. This can be done by direct calculation of dynamic responses or by applying some dynamic amplification factor to the response and stresses obtained from a static wave response analysis.

For the storm condition where harmonic loads with periods between 14 and 19 seconds are applied to the structure, the most efficient way is to use the second approach as these periods are far enough from structural resonance for the method to give reasonable results. More difficulties arise however, when fatigue is assessed using shorter period waves representative of the wave climate for the region of interest.

D.a.f's are calculated bearing in mind the following points:-

1. A d.a.f. can only strictly be associated with a particular mode shape and frequency and is calculated by extension of the one degree of freedom idealisation given by:-

$$\gamma(T,T_n) = \frac{\text{Dynamic Deflection}}{\text{Static Deflection}}$$

$$= \left\{ \left(1-\left(\frac{T_n}{T}\right)^2\right)^2 + \left(2\xi_n \frac{T_n}{T}\right)^2 \right\}^{-\frac{1}{2}} \quad \text{.... 1)}$$

or alternatively written in terms of frequencies

$$\gamma(\omega,\omega_n) = \left\{ \left(1-\left(\frac{\omega}{\omega_n}\right)^2\right)^2 + \left(2\xi_n \frac{\omega}{\omega_n}\right)^2 \right\}^{-\frac{1}{2}} \quad \text{.... 2)}$$

where T is the time period of the forcing function (e.g. wave period)

ξ_n is the proportion of critical damping associated with the n^{th} mode shape.

T_n is the natural period of the vibration associated with the n^{th} mode shape.

The damping value ξ_n will vary with the mode considered. Usually for welded steel jackets under wave loading ξ_n is taken to be 0.02 for all modes and includes hydrodynamic damping [1].

It is understandable that the d.a.f. for a one degree of freedom system, Figure 2, can safely be applied to both displacements and member end stresses to account for dynamic effects. For frames such as those in Figure 1 it is less likely that the same factor can be applied to member end stresses and nodal or joint displacements in all degrees of freedom. This is why a direct comparison of static and dynamic stress levels has been performed. (See Section 3).

2. The fundamental sway mode of a steel jacket in water corresponds closely to the global response shape under long wave loading. It is this circumstance that has enabled designers to apply the above formula as a global d.a.f. in all cases of wave loading.

For local vibration of part of the structure such as out of plane vibration of a conductor bracing level, then a d.a.f. with T_n equal to the natural period of local vibration is appropriate for that part of the stress range caused by local hydrodynamic loads.

It is the "frame action" component of member stresses which should be amplified according to equations 1 and 2 with T_n = sway natural period. In practice this factor is applied conservatively to the total member stresses due to environmental loads.

The relative contribution of frame action effects to total stress range is evaluated in Section 4.

3. The storm wave represents an extreme sea state which contains a wide range of superposed wave periods. Some of these periods are closer to structural resonance and therefore may be associated with a higher d.a.f. However, d.a.f's are generally computed from the storm wave period representative of the sea state and which is considered to represent the overall behaviour. This will be unconservative in most instances.

This point is addressed by the construction of a special d.a.f. which takes into account the frequency spread of the sea state represented by the given wave. Its derivation is dealt with below.

2.1 Spectral Dynamic Amplification Factors

Some method of representing the dynamic amplification corresponding to the sea state represented by the design waves must be used. This is done in the following way [2].

For a representative degree of freedom (e.g. horizontal deflection of the deck) calculate the standard deviation of the dynamic response σ_d. This is given by:-

$$\sigma_d^2 = \int_0^\infty S_{\eta\eta}(\omega) \left| T_{dyn}(\omega) \right|^2 d\omega \quad \text{.... 3)}$$

where T_{dyn} is the dynamic transfer function for the chosen degree of freedom for the structure considered and is calculated by the dynamic response to a series of linear waves with differing periods.

$S_{\eta\eta}(\omega)$ is the spectral form for the sea state considered.

ω is the angular frequency.

A suitable form for $S_{\eta\eta}(\omega)$ is the Pierson Moskowitz spectrum given by:-

$$S_{\eta\eta}(\omega) = \frac{H_s^2 T_c}{4\pi} \left(\frac{T_c\,\omega}{2\pi}\right)^{-5} \exp\left[-\frac{1}{\pi}\left(\frac{\omega T_c}{2\pi}\right)^{-4}\right] \quad \text{. 4)}$$

where T_o is the zero crossing period for the sea state

and Hs is the significant wave height for the sea state.

It is assumed that for a 12 hour storm,

$$T_o = T \times 0.81 \quad \text{.... 5)}$$

$$\& \; H_s = H/1.8 \quad \text{.... 6)}$$

where H and T are the height and periods of the representative wave.

Now assume that the dynamic response for each frequency component may be written in terms of the conventional d.a.f. γ (equation 2) and the static response, then:-

$$T_{dyn}(\omega) = T_{stat}(\omega)\,\gamma(\omega,\omega_n) \quad \ldots.\ 7)$$

where ω_n is the natural angular frequency of the relevant sway mode.

Then the static standard deviation of response σ_s is given by

$$\sigma_s^2 = \int_0^\infty S_{\eta\eta}(\omega)\,|T_{stat}(\omega)|^2\,d\omega \quad \ldots.\ 8)$$

hence the ratio of the dynamic to the static standard deviations, the spectral d.a.f. γ_{spect} is given by:-

$$\gamma_{spect}^2 = \frac{\sigma_d^2}{\sigma_s^2} = \frac{\int_0^\infty S_{\eta\eta}(\omega)|T_{stat}(\omega)|^2\gamma^2(\omega,\omega_n)\,d\omega}{\int_0^\infty S_{\eta\eta}(\omega)|T_{stat}(\omega)|^2 d\omega} \quad 9)$$

Evaluation of this expression requires detailed knowledge of the form of the transfer function Tstat (ω).

A fair estimate of the spectral dynamic amplification factor may be obtained by assuming that the function Tstat is flat over the peak of the wave energy spectrum $S_{\eta\eta}$ ( ).

Cancellation of this function now becomes possible and:

$$\gamma_{spect}^2 = \frac{\int_0^\infty S_{\eta\eta}(\omega)\,\gamma^2(\omega,\omega_n)\,d\omega}{\int_0^\infty S_{\eta\eta}(\omega)\,d\omega} \quad \ldots.\ 10)$$

This is now a dynamic amplification factor which takes due account of the spread of frequencies represented by the design wave and their dynamic interaction with the structure. This factor can be applied without prior knowledge of the transfer function Tstat for the structure.

This spectral d.a.f. is compared with the conventional d.a.f. for a range of sea states in Figure 4a for a one degree of freedom system with a natural period of 3 seconds. The design period is that period corresponding to the design wave for each sea-state, here taken to be To/0.81. In the range of interest (4 to 17 seconds) the spectral d.a.f. lies above the one degree of freedom d.a.f. commonly used. The difference being important for waves used in fatigue assessment.

The situation for a structure with natural period 6 seconds is somewhat worse (figure 4b). Care must however be taken in assuming a flat transfer function for the structure for the moderate sea states and hence for design periods below 7 or 8 seconds. This topic is still under investigation.

3.0 COMPARISON OF STATIC AND DYNAMIC RESPONSE

The wave cases shown in Table 2 were applied in three directions to the computer models of jacket 1 and jacket 2 and nodal loads derived using the Morison equation formulation in the ASAS suite of programs.

These loads were then applied to the jacket stiffness models and static member end loads and moments derived.

Modal analyses were performed on both dynamic models which contained mass and added mass distribution information for the structures, to produce 30 mode shapes and frequencies. The modal frequencies are given in Table 3.

Selected mode shapes are shown in figure 5.

The dynamic response to the waves shown in Table 2 was then calculated by model superposition using the results of the static analyses to correct for higher modes by static backsubstitution. A damping value of 0.02 (2% of critical) was used for all applied waves, 5% was also used for selected cases.

The axial forces in the legs due to loading from waves of height 1m at different periods are shown in figures 6a and 6b. Figure 6a shows the variation of axial load in the legs of Jacket 1 from static analysis. Figure 6b shows the corresponding quantities from the full dynamic analysis with static backsubstitution.

The axial forces are monotonically decreasing with height above the sea bed as would be expected. For waves with periods at or below structural resonance, substantial dynamic amplification occurs. In the legs the correction associated with static backsubstitution is minor (only 7% at resonance) as the load values are largely determined by the global response of the jacket. The same general behaviour was observed for jacket 2. The general high level of load at the base of the structure indicates the importance of correct dynamic amplification at these locations.

Figures 7a and 7b show the d.a.f.'s associated with wave loading for 9 second waves with heights from 1 to 12 metres for both jackets, these d.a.f's were obtained from direct calculation of static and dynamic response. In general there is little variation with wave height at the base of the structure where the conventional d.a.f is conservative.

Towards the still water level in both cases there is an increase in d.a.f. over and above the conventional value, this effect decreases with increasing wave height bringing the amplification on the higher load values closer to their expected value. It should also be borne in mind that the wave induced loads in this region of the structure will be considerably less than those nearer the base (figures 6a, 6b).

Figures 8a and 8b show the corresponding results for the axial brace loads for both structures. Again there is little variation with wave height with the conventional d.a.f. giving conservative results except at the lower levels. The unconservatism in this case is eliminated by the use of the spectral d.a.f. described above. The oscillation of the curve in figure 8b is a result of the framing pattern.

Figure 9 shows the variation of d.a.f. for waves of unit height with periods from 2.5 to 17 seconds for jacket 2 (Jacket 1 similar). The leg axial loads were used. The conventional d.a.f's are given in brackets with the equivalent spectral d.a.f. value. Waves near resonance indicate again the unconservatism in the conventional d.a.f. There is again a noticeable increase in d.a.f. near the still water level.

The effect of various damping coefficients on the dynamic results was also examined for a number of locations on jacket 1. Percentages of critical damping of 2% and 5% were examined and member end loads compared. If F2 is the member end force for 2% damping and F5 is the member end force for 5% damping then:

$$\frac{F2}{F5} = \frac{\gamma 2}{\gamma 5}\frac{Fs}{Fs} = \frac{\gamma 2}{\gamma 5} \qquad \text{.... 11)}$$

where Fs is the corresponding static value and $\gamma 2$ and $\gamma 5$ are the d.a.f's for 2 and 5% damping. This ratio may be easily calculated from equations 1 and 2.

Generally near resonance (sway mode) the ratio actually derived was much larger than that calculated i.e. the effect of damping reduction was higher than expected. This effect was observed at all stations up the jacket. Waves at about 5 seconds did not exhibit this anomoly and so in design for strength and fatigue resistance this effect is not important.

4.0 FRAME ACTION AND DYNAMIC AMPLIFICATION

In Section 2.0 reference was made to the importance of the application of the correct d.a.f. to the statically derived stresses bearing in mind the relevant mode for that part of the structure. In practice the d.a.f. corresponding to the sway mode period is applied to the total stresses/displacements. This is conservative as this d.a.f. is derived for the longest period and is hence greater than d.a.f's for lower periods for wave loading above sway mode resonance.

This sway mode component of response corresponds closely to the frame action response of the structure and so the component of stress caused by frame action can safely be amplified by the conventional d.a.f. The component of stress caused by local hydrodynamic loading should be amplified by the d.a.f. corresponding to local vibration at a lower natural period. Hence:-

$$S_d = \gamma_1 S_f + \gamma_n S_L < \gamma_1 (S_f + S_L) = \gamma_1 S \qquad \text{.... 12)}$$

as $\gamma_n < \gamma_1$ above global resonance.

where γ_1 is the d.a.f. for the fundament sway mode
S_f is the stress caused by frame action
γ_n is the d.a.f for the natural vibration of the local structure
S_L is the stress from local hydrodynamic loading
and S is the total statically derived stress.

Figures 8a, 8b and 9 exhibit the expected behaviour in that the d.a.f's associated with positions near the base of the structure are largely associated with frame action only and hence the larger d.a.f. from the sway mode is applicable here.

In order to quantify this effect the member end loads for the members indicated in figures 10a and 10b were determined with and without local hydrodynamic loading.

This was achieved by setting the hydrodynamic coefficients to zero for these members to achieve a comparison. The assumption that the overall response with and without local loading on these members was virtually unchanged and was born out by the results.

It was found that the greatest variation between member end loads was found for waves travelling perpendicular to the horizontal brace orientations. Waves of heights 1, 3, 6, 9 and 12m were run with a wave period of 9 seconds for both jackets.

In Figures 11a and 11b the variation of frame action and wave induced axial loads are given for horizontal members near the splash zone. The same qualitative behaviour was found for the other members considered. The frame action component increases nearly linearly with wave height as does the total load. The slope of the curve for local loading, however, is somewhat less than that for frame action reflecting the cumulative nature of frame action loading. For waves above 3m the direct hydrodynamic loading was found to contribute less than 15% to the total.

Hence the conservatism implicit in the conventional approach is not large or excessive but appears to be more significant for the deeper water structure in spite of the larger number of members in this structure.

5.0 THE CHOICE OF FATIGUE WAVES

In order to accurately represent a wave climate in a deterministic fatigue analysis it is necessary to choose a small number of characteristic waves, each of which results in a proportion of the total fatigue damage. Ideally each characteristic wave should give rise to an equal proportion of the damage.

5.1 The Choice of Fatigue Waves

The selection of wave heights is based on the following assumptions:-

1. Stress range (S) is proportional to H^{α}
2. Number of cycles to failure (N) is proportional to S^{m} where m = 'slope' of SN curve and is taken to be equal to 3 for the D.En. 'T' curve.

Therefore $N \sim S^{-3} \sim H^{-3\alpha}$ .... 13)

3. Damage is the ratio of number of occurrences to the number of cycles to failure (N).

$$D = n/N \sim nH^{3\alpha} = nS^3 \qquad \text{.... 14)}$$

This process is illustrated in figure 12. The cumulative damage curve may be used to define the wave height ranges represented by each wave. The number of cycles may be obtained from an exceedance diagram for the area. Particular care must be taken in choosing H1 in the lowest interval because of the large number of cycles here and the changing slope of the cumulative damage curve. This may be achieved by subdividing this interval and iteratively calculating H1 to give equal damage to that given by all waves in the range.

The crucial parameter is α which is usually taken to be 1.8. Our results indicate that a value of α of 1.025 gives the best fit. Figures 13a and 13b show this, as for varying wave heights (at 9 seconds) the curves of variation of stress/H^{α} lie on top of one another for this value.

5.2 Dynamic Amplification Factors for Fatigue

Dynamic amplification has two major effects:-

1. The damage distribution shown in figure 12a is biased towards global structural resonance, and hence the curve is displaced toward lower wave heights and periods giving different representative waves.

2. The damage associated with each wave height range will be amplified which must then be reflected in an appropriate dynamic amplification factor for each representative wave. This amplification factor will not necessarily be the same as the conventional d.a.f which would normally be associated with each representative wave height.

As dynamic stress range S_d is related to the statically derived stress range S by:-

$$S_d = S\,\gamma \qquad \text{.... 15)}$$

with γ being evaluated at each wave period,

then the damage incorporating dynamic effects Dd has the following functional form:-

$$D_d = \frac{n}{N} \sim n(\gamma S)^3 = nS^3\gamma^3 = D\gamma^3 \qquad \text{.... 16)}$$

hence point by point

$$\gamma = \left(\frac{D_d}{D}\right)^{1/3} \qquad \text{.... 17)}$$

(for the 'T' curve with 'slope' 3)

The cumulative damage curve may now be constructed as before and will be of similar form to figure 12b but again displaced to lower wave heights and periods.

The representative wave heights will now be lower than their statically derived equivalents.

If di is the cumulative damage up to the upper end of wave height range i and d'i is the same cumulative damage incorporating dynamic amplification, then the damage associated with range i is given by:

$$d_i - d_{i-1}$$

and with dynamic amplification

$$d'_i - d'_{i-1}$$

Taking the ranges and representative wave heights derived from the dynamic cumulative curve.

$$\frac{\text{Damage for range i (dynamic)}}{\text{Damage for range i (static}} = \frac{d'_i - d'_{i-1}}{d_i - d_{i-1}} \qquad \text{.... 18)}$$

Given that the damage amplification is proportional to the cube of the amplification of stress range we may define a dynamic amplification factor γ_i for each range in the following way:-

$$\gamma_i^3 = \frac{d'_i - d'_{i-1}}{d_i - d_{i-1}} \qquad \text{.... 19)}$$

These 'fatigue' dynamic amplification factors γ_i will now fully represent the effect of dynamic amplification across each range.

These fatigue d.a.f's depend in a complex way on the cumulative influence of γ over the range and hence on structure sway natural period.

In Table 4 the fatigue d.a.f's for jacket 1 are compared with the conventional d.a.f. for the representative waves appropriate for a fatigue analysis of this structure.

In this table the first column shows the representative wave heights derived from static considerations only, the second column contains the representative waves taking into account dynamic amplification. The representative wave for the first interval is greatly reduced reflecting the bias towards structural resonance. The fatigue d.a.f. is larger than would be calculated for the period corresponding to this wave reflecting the increased amplification for waves near resonance.

In practice during design a cut-off frequency would be derived to represent fatigue thresholds and short-crestedness associated with short waves.

It becomes even more crucial to use this approach for structures with higher fundamental natural periods as the conventional approach will vastly overestimate the damage associated with representative waves near structural resonance.

6.0 CONCLUSIONS

Two structures with comparable fundamental sway periods have been examined, one in 150m of water and one in 250m of water, to investigate the effect of water depth on dynamic amplification.

Provided due care is taken to represent the spread of frequencies in a design sea state a spectral dynamic amplification factor may be safely used to correct the results of a static analysis.

Frame action loads are found to dominate in both cases and hence a d.a.f. based on the fundamental sway period is adequate.

Plots of the actual dynamic amplification exhibit little variation with wave height and for the higher waves little variation from the sea-bed to the topsides.

Investigation of the relatioship between stress range S and wave height H indicates that S   H 1.025.

A fatigue d.a.f. has been postulated which is based on damage amplification, this again takes into account the spread of wave frequencies in the range represented by each wave used in a deterministic fatigue analysis.

With proper care, reliable dynamic amplification factors may be used to correct static analyses of realistic structures in water depths as high as 250m.

7.0 REFERENCES

1. "DnV Rules for the Design Construction and Inspection of Fixed Offshore Structures 1977". Appendix G Dynamic Analysis.

2. Brebbia C.A and Walker S. "Dynamic Analysis of Offshore Structures" Chapters 3 and 4, Newnes Butterworhts, 1980.

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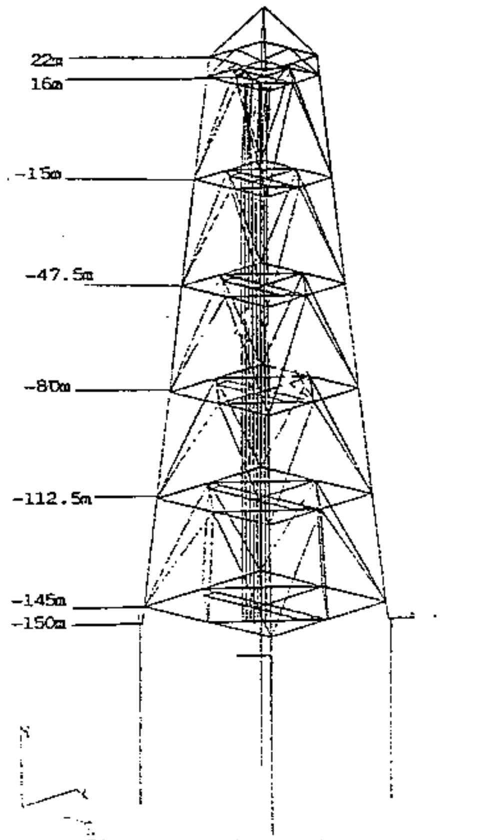


Figure 1a Jacket1 150m Water Depth

NORTHERN NORTH SEA WAVE DATA

for the period Mar. 1973 to Feb. 1980 (excluding 1979)

Units indicated in parts per thousand. Total observations =16042

significant wave height Hs (m) \ mean wave period Tz (s)	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14	14–15	15–16
11.5–12.0										*	*					
11.0–11.5										*	*					
10.5–11.0											*					
10.0–10.5											*	*	*			
9.5–10.0								*	*		*	*				
9.0–9.5									*	*	*	*				
8.5–9.0								*	*	1	1	*				
8.0–8.5									*	1	*	*				
7.5–8.0								*	*	4	*	*	*			
7.0–7.5								*	1	4	1	*				
6.5–7.0								*	3	4	*	*				
6.0–6.5							*	1	7	3	*	*	*			
5.5–6.0							*	4	11	5	*	*	*			
5.0–5.5						*	1	10	11	4	*	*	*	*		
4.5–5.0					*	*	4	20	14	4	*	*	*			
4.0–4.5						*	14	30	12	2	1	*	*			
3.5–4.0				*		4	29	24	9	2	*	*	*			
3.0–3.5				*	*	15	46	27	10	2	*	*	*			
2.5–3.0			*	*	4	42	49	22	8	2	*	*	*			
2.0–2.5			*	2	21	52	46	22	8	3	1	*				
1.5–2.0			*	5	48	45	34	19	8	4	1	*				
1.0–1.5			*	12	42	38	27	12	5	3	*	*				
0.5–1.0				7	14	9	7	3	2	*	*	*				
0.0–0.5					*	*	*	*	*							

Figure 2 Scatter Diagram

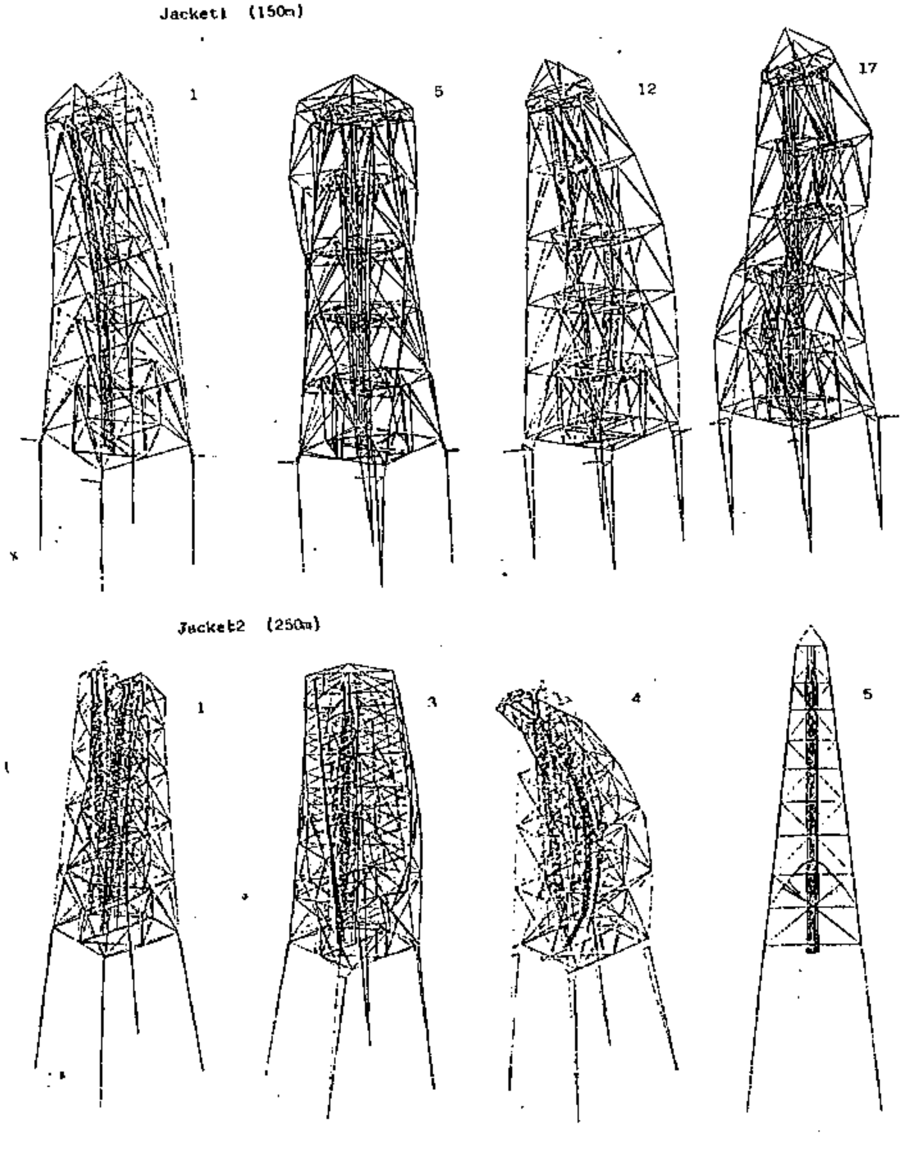


Figure 5 Selected Mode Shapes

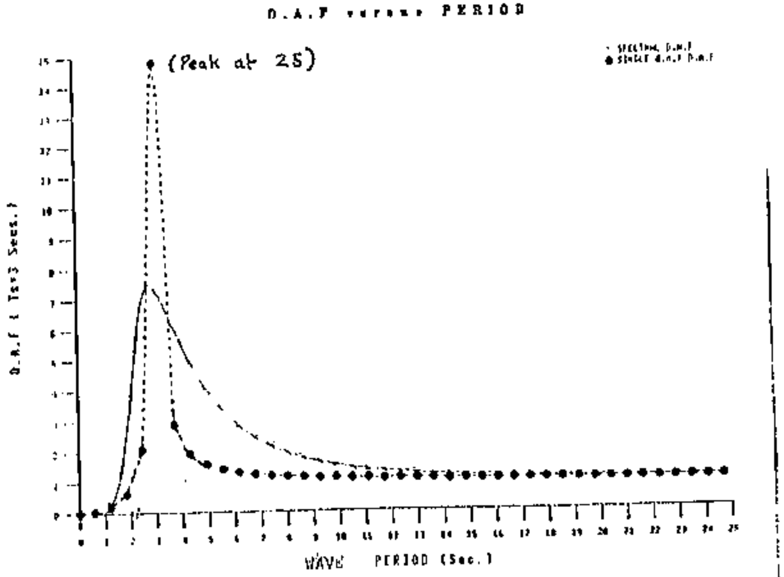


Figure 4a Spectral Dynamic Amplification Factor (Ts=3 secs.)

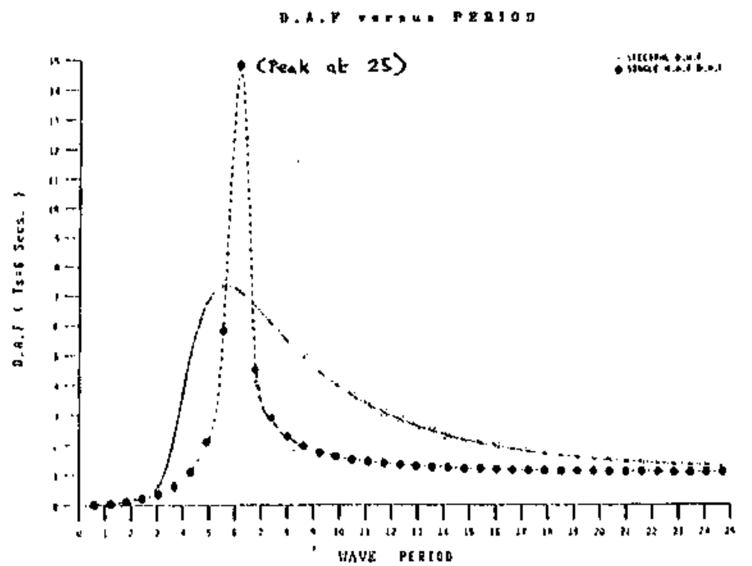


Figure 4b Spectral Dynamic Amplification Factor (Ts=6 secs)

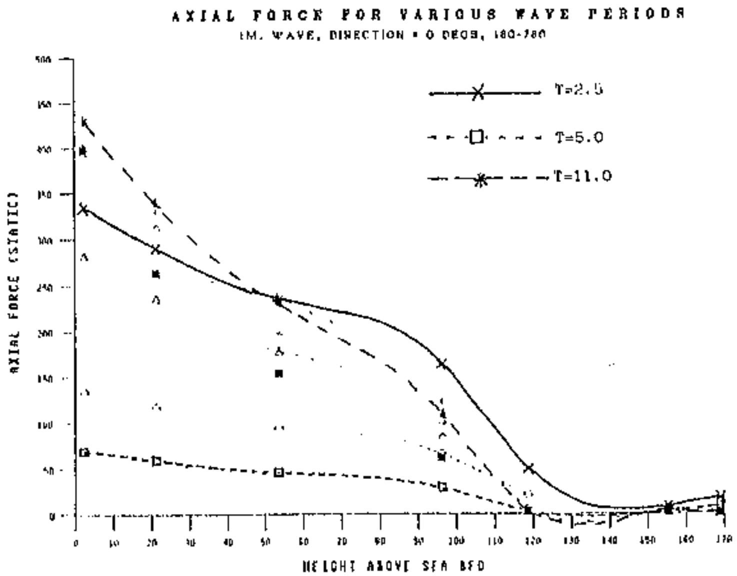


Figure 6a Axial Forces in Legs Jacket1 (Static)

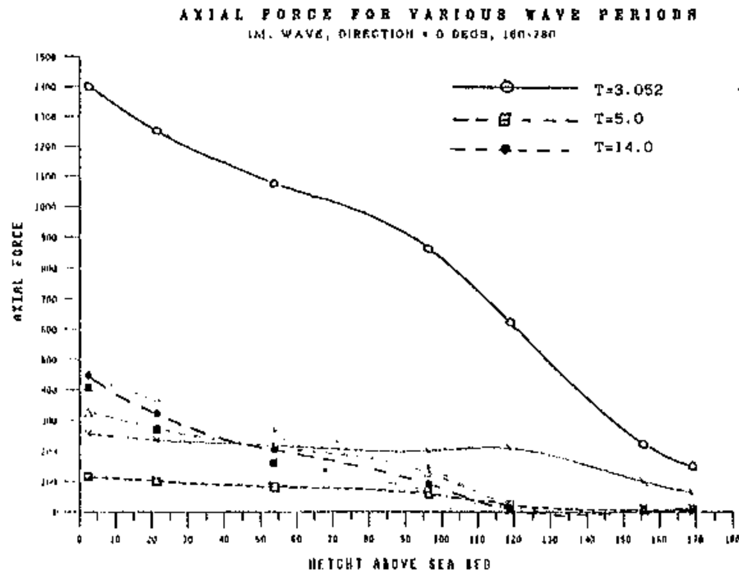


Figure 6b Axial Forces in Legs Jacket1 (Dynamic)

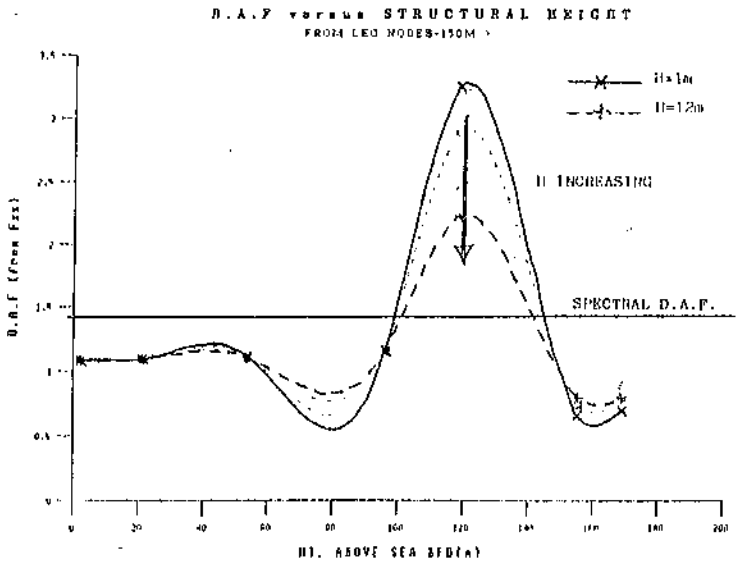


Figure 7a Dynamic Amplification Factor (9 second wave) Jacket1

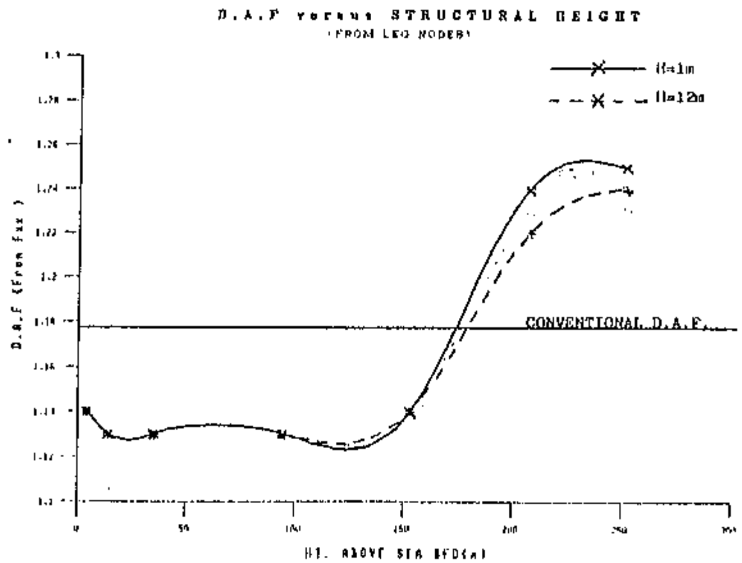


Figure 7b Dynamic Amplification Factor (9 second wave) Jacket2

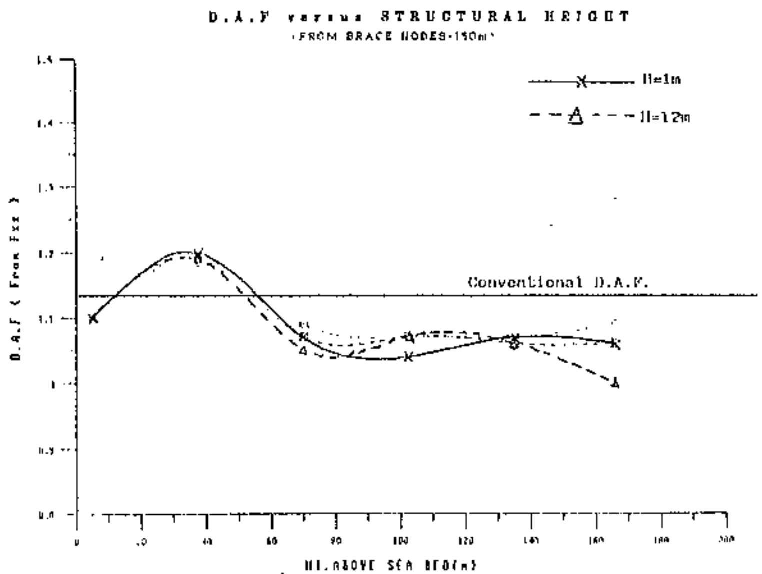


Figure 8a Variation of D.A.F. from brace axial forces Jacket1

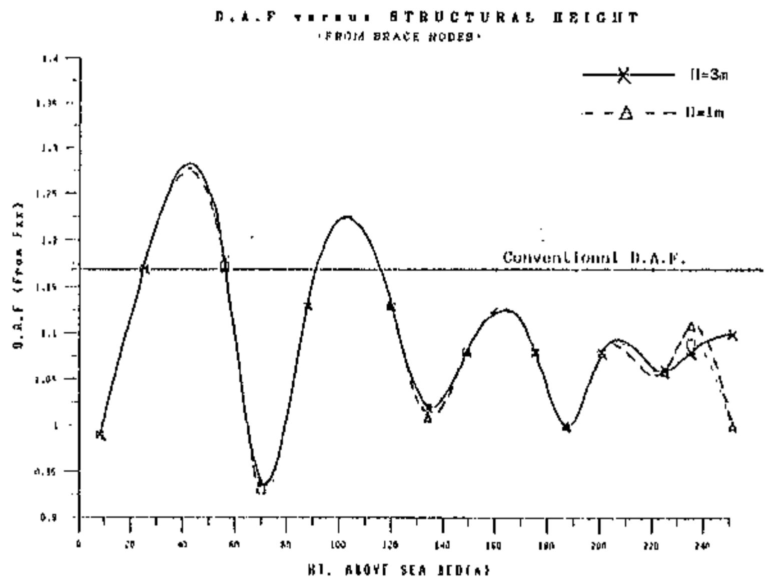


Figure 8b D.A.F. From brace axial forces (9 second wave) Jacket2

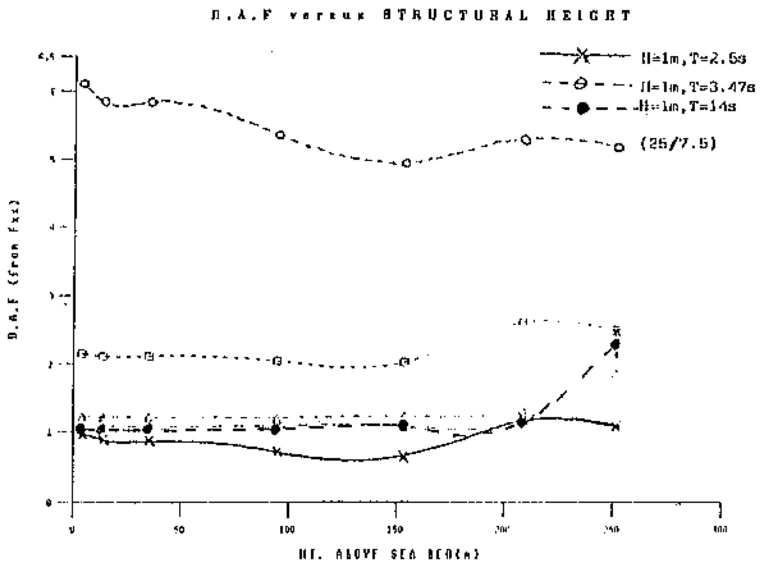
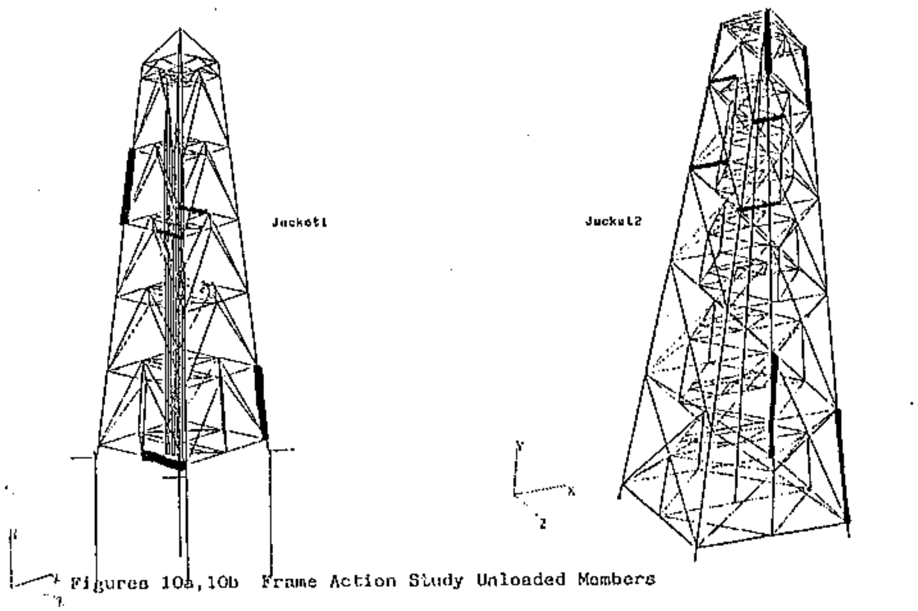


Figure 9 D.A.F. Vs. Wave Period and Height above mud line Jacket2



Figures 10a,10b Frame Action Study Unloaded Members

Figure 11a FRAME ACTION & WAVE FORCE Vs WAVE HEIGHT

FOR MEMBER 507 (150 Jacket)

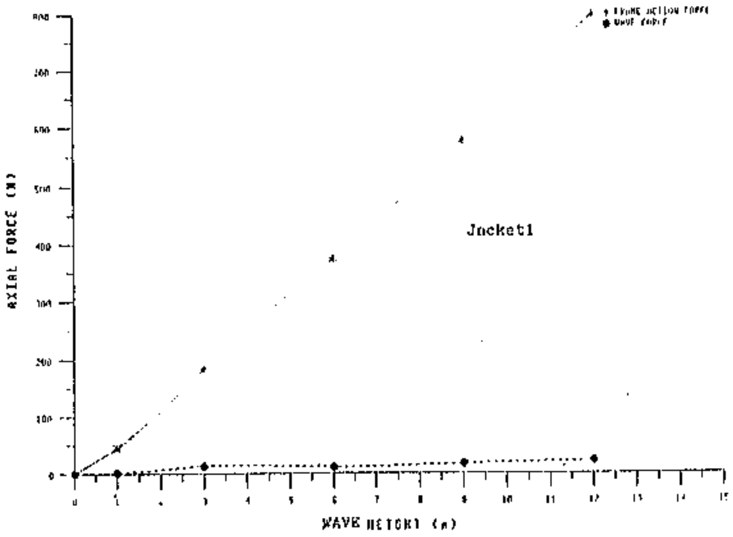
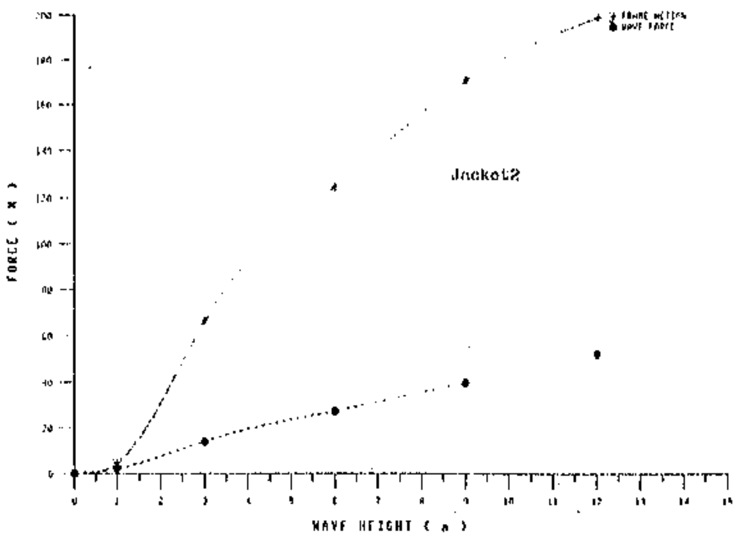


Figure 11b FRAME ACTION & WAVE FORCE vs WAVE HEIGHT

FOR MEMBER 906 (250 m Jacket)



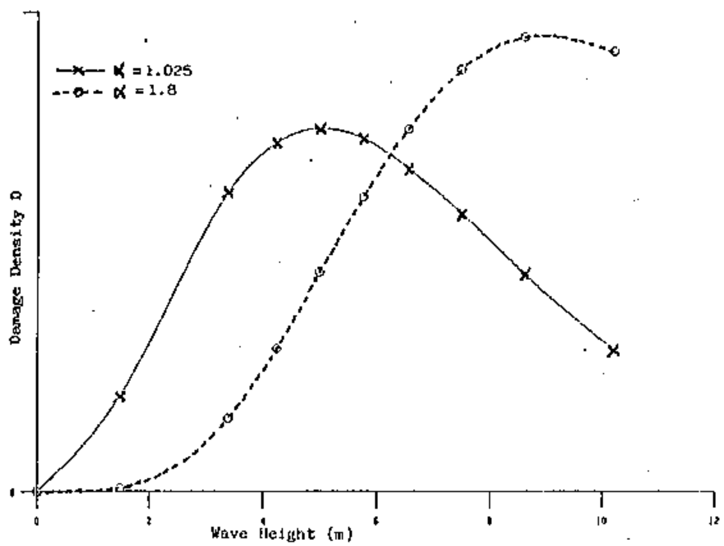


Figure 12a Typical Damage Distribution Against Wave Height (Static)

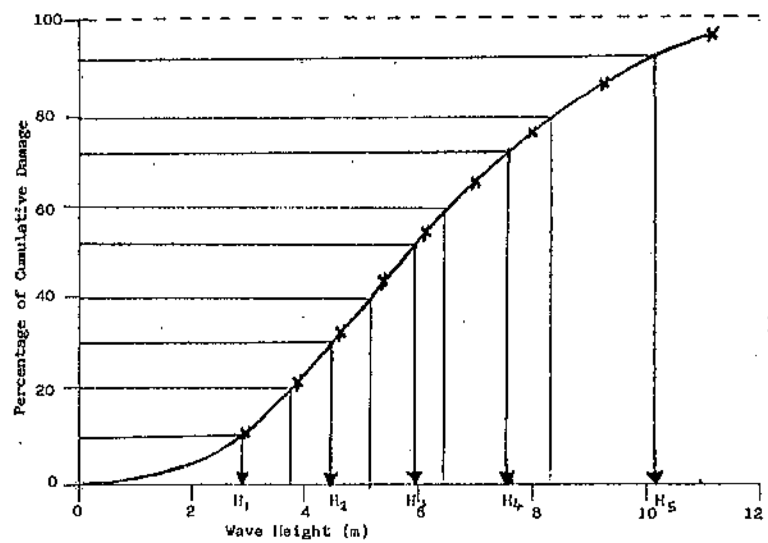


Figure 12b Cumulative Damage Distribution/ Choice of Fatigue Waves

Figure 13b STRESS-WAVE FUNCTION Vs STRUCTURAL HT.

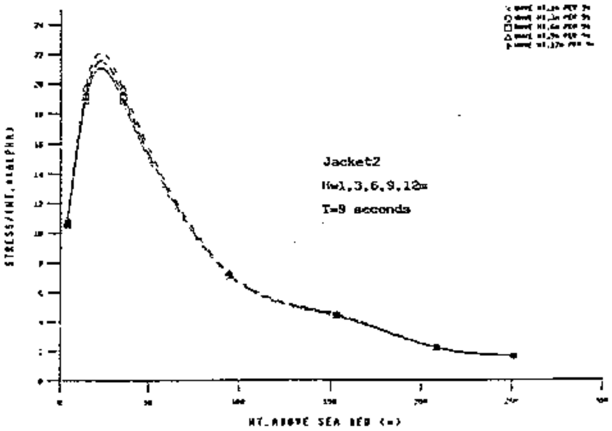


Figure 13a

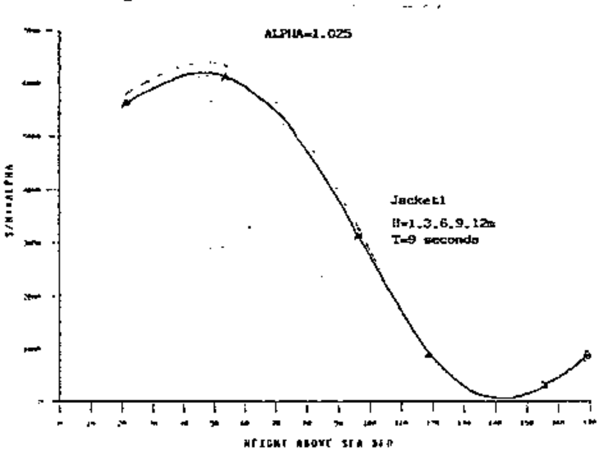


TABLE 1 ENVIRONMENTAL CONDITIONS

Water Depth — Jacket 1: 150m LAT; Jacket 2: 250m LAT

Waves

Wave Occurrences (30 year period)

Wave ht (m)	Period (s)	Total Number of Cycles
0-1	5.5	66,880,000
1-2	6.4	35,370,000
2-3	7.4	18,850,000
3-4	8.6	10,100,000
4-5	9.5	5,447,000
5-6	9.9	2,949,000
6-7	10.5	1,602,000
7-8	10.8	873,500
8-9	11.2	477,400
9-10	11.6	261,700
10-11	11.9	143,700
11-12	12.2	79,050
12-13	12.6	43,560
13-14	12.9	24,040
14-15	13.2	13,290
15-16	13.5	7,349
16-17	13.8	4,070
17-18	14.1	2,256
18-19	14.4	1,251
19-20	14.7	695
20-21	15.0	386
21-22	15.2	215
22-23	15.5	121
23-24	15.8	68
24-25	16.0	36
25-26	16.3	20
26-27	16.5	10
27-28	16.8	7
28-29	17.0	3
30 yr Hmax (3 hr)		28.8
TOTAL		143,100,000

Marine Growth

50mm thickness on radius will be assumed between elevations +1.5m and -40m with reference to L.A.T.

Currents and Winds

As the emphasis is on dynamic effects no currents or winds will be considered.

TABLE 4 FATIGUE WAVES AND D.A.F's

Static Wave Height	Dynamic Wave Height (m)	Period (s)	No of Cycles	Fatigue d.a.f	Conventional d.a.f.
2.93	1.94	4.34	1.301×10^8	3.36	2.01
4.63	4.55	6.33	7.851×10^6	1.24	1.25
6.15	6.09	7.99	3.219×10^6	1.27	1.17
7.99	7.93	9.01	1.437×10^6	1.13	1.13
11.17	11.16	10.70	4.975×10^5	1.08	1.08

TABLE 2 WAVE CASES EXAMINED (BOTH JACKETS)

Wave Height (m)	Wave Period														
	2.5	3.02	4	5	6	7	8	9	10	11	14	15	16	17	17.7
1	X	X	S	X		X		X		X	X			X	
3					S			X							
6							S	X							
8								S							
9								X							
10									S						
12								X							
14											S				
18												S			
20													S		
30														S	

TABLE 3 RESULTS OF MODAL ANALYSES

	Jacket 1 - 150m water		Jacket 2 - 250m water	
Mode	Period(sec)	Description	Period(sec)	Description
1.	3.052	Global Sway	3.473	Global Sway
2.	3.009	Global Sway	3.415	Global Sway
3.	1.213	Global tension	2.027	Global torsion
4.	1.086	Torsion top bay	1.907	Global bending
5.	1.086	Global torsion	1.899	Global bending
6.	1.085	Local top bay	1.683	Global bending
7.	1.085	Bending upper bays	1.603	Local top bay
8.	1.085	Conductor bracing levels	1.603	Local top bay
9.	1.084	Torsion upper bays	1.602	Local top bay
10.	1.083	Torsion upper bays	1.602	Local top bay
11.	1.047	Torsion upper bays	1.602	Local top bay
12.	0.982	Global bending	1.602	Local top bay
13.	0.969	Global bending	1.601	Local top bay
14.	0.789	Local topside	1.554	Conductor bracing levels
15.	0.665	Torsion top bays	1.466	Conductor bracing levels
16.	0.624	Local topside	1.375	Conductor bracing levels
17.	0.575	Global bending	1.117	Conductor bracing levels
18.	0.565	Global bending	1.107	Conductor bracing levels
19.	0.564	Global bending	1.090	Conductor bracing levels
20.	0.564	Global bending	0.987	Local modes (various)
21.	0.564	Global bending	0.951	Local modes
22.	0.564	Top bay bending	0.932	Local modes
23.	0.564	Top bay bending	0.917	Local modes
24.	0.563	Global bending	0.909	Local modes
25.	0.563	Global bending	0.908	Local modes
26.	0.544	Conductor bracing	0.900	Local modes
27.	0.513	Conductor bracing	0.871	Local modes
28.	0.415	Double torsion	0.870	Local modes
29.	0.406	Double torsion	0.849	Local modes
30.	0.406	Double torsion	0.821	Local modes