ECONOMIC DYNAMIC ANALYSIS OF OFFSHORE STRUCTURES

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ABSTRACT

Methods are described which enable dynamic effects to be accurately incorporated into a static structural analysis by the application of dynamic amplification factors (d.a.f's).

A spectral dynamic amplification factor (d.a.f.) is derived which takes into account the frequency spread of the sea states represented by design waves.

Consideration of frame action loading (or loading due to global structural response) indicates that for members near the splash zone a d.a.f. based on a combination of local and global natural period is appropriate.

A fatigue d.a.f. is postulated which avoids the over-estimation of fatigue damage usually attributed to small, short waves.

Two realistic designs for fixed offshore structures in 150m and 250m of water are examined to investigate the effects of dynamic amplification.

Particular reference is made to the extension of conventional d.a.f.'s to deep-water structural design.

NOMENCLATURE

- γ One degree of freedom dynamic amplification factor (d.a.f.).
- T Forcing or wave period
- Tn Natural structural period associated with the nth mode
- T1,T2 Natural sway or fundamental periods of jacket
- ω Forcing or wave angular frequency (radians/sec)
- ωn Natural frequency associated with nth mode (nth eigen value)
- ξn Proportion of initial damping associated with nth mode
- σd Standard deviation of dynamic response
- $S_{\eta\eta}(\omega)$ Wave energy spectrum (one-sided)
- Tdyn(ω) Transfer function of dynamic response
- Hs Significant wave height
- T_3 Zero crossing period
- H wave height
- Tstat (ω) Transfer function of static response
- σs Standard deviation of static response
- γspect Spectral dynamic amplification factor
- S Statically derived stress range
- S_L Stress range from local hydrodynamic loading
- Sf Stress range caused by frame action
- $\gamma 1$ d.a.f. for fundamental sway mode (= γ)
- γn d.a.f for natural vibration of local structure
- α Exponent to be applied to wave height to give variation of stress range
- m Slope of the S-N fatigue curve
- N Number of cycles to failure from S-N curve
- n Number of cycles at stress range S
- D Damage density - damage at stress range S from wave height H
- Sd Stress range including dynamic effects
- Dd Damage incorporating dynamic effects
- di Cumulative damage to upper bound of wave height range "i"
- γi Fatigue dynamic amplification factor associated with wave height range "i".
- Hi Representative wave height for wave height range "i".
- x Horizontal displacement of topsides centre of gravity
- F(z) Distributed loading on jacket due to wave
- F' Equivalent generalised force
- Z Elevation above sea bed
- m' Generalised mass associated with x
- m Mass plus added mass per unit length of the substructure
- K' Generalised stiffness associated with x
- $\underline{M}$ Mass matrix of jacket
- $\underline{C}$ Damping matrix
- $\underline{K}$ Jacket stiffness matrix
- $\underline{X}$ Node Displacement/rotation vector
- $\underline{F}$ Load vector
- mi,ci,ki Generalised mass, damping and stiffness associated with generalised displacement xi
- fi Generalised force for ith mode.

1.0 INTRODUCTION

In 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 economically by using a static approach and applying dynamic amplification factors (d.a.f.'s) to the member stresses derived.

Extensions of the conventional single degree of freedom (d.a.f.) representation necessary for jackets in deeper waters are described.

1. The spectral d.a.f. γ spect which takes into account the frequency spread in the sea state represented by design waves.

2. The fatigue d.a.f. γi which correctly amplifies the damage associated with representative fatigue waves.

Two actual jacket designs are used to evaluate the impact of dynamic effects on 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 backsubstitution was used to correct for the higher modes (above mode 30) not explicitly derived.

The environmental conditions assumed are given in Table 1. The waves examined for both structures are given in Table 2. Realistic pile spring stiffnesses were used.

The topside mass of each jacket was taken to be 17,000 tonnes giving a fundamental sway period for jacket 1 of 3.02 secs and for jacket 2 a fundamental period of 3.47 secs. The two periods were arranged to be similar in order to isolate any effects associated with water depth.

The main aim of this paper is to quantify in realistic circumstances the contribution of global sway response or frame action and dynamic effects to local stress levels and fatigue damage. These methods are applied by John Brown in the design of actual structures for the North Sea.

2.0 ONE DEGREE OF FREEDOM APPROXIMATION

2.1 General Remarks

Implicit in the use of the conventional dynamic amplification factor γ is the assumption that the response of the structure to wave loading may be closely represented by an equivalent one degree of freedom system.

If we chose the horizontal displacement of the centre of gravity of the topsides x as a representative or generalised degree of freedom and define a generalised force f' to represent the distributed loading F(z) due to an incoming wave by integrating wave force moments about the sea bed, then a system shown in Figure 2 may be used to represent the global platform response.

If m' is the generalised mass taken to be M+0.23ml where M is the topside mass and m is the mass plus added mass per unit length of the jacket over a length l, we may define K' to be the generalised stiffness defined numerically as that force which will give unit displacement at the topside centre of gravity.

This equivalent continuous system may then be mathematically represented by the spring, mass system in the lower part of Figure 2. The constant C represents the contributions of structural and hydrodynamic damping usually expressed in terms of the proportion of critical damping and is taken to be 0.02 for welded steel structures in sea-water.

$$\xi = \frac{C}{2\sqrt{m'K'}} \qquad \ldots 1)$$

The natural frequency of this system is given by:-

$$\omega_1 = \sqrt{\frac{K'}{m'}} \simeq \sqrt{\frac{K'}{M + 0.23ml}} \qquad \ldots 2)$$

K may be derived from a static stiffness analysis of the jacket computer model.

The equation of motion of the idealised system is given by:-

$$m'\ddot{x} + c'\dot{x} + K'x = F' \qquad \ldots 3)$$

or for harmonic loading at frequency ω

$$|x| = \frac{|F'|}{K'}\left\{\left(1-\left(\frac{\omega}{\omega_1}\right)^2\right)^2 + \left(2\xi\frac{\omega}{\omega_1}\right)^2\right\}^{-\frac{1}{2}} = \frac{|F'|}{K'}\gamma \qquad \ldots 4)$$

or in terms of periods:-

$$|x| = \frac{|F'|}{K'}\left\{\left(1-\left(\frac{T_1}{T}\right)^2\right)^2 + \left(2\xi\frac{T_1}{T}\right)^2\right\}^{-\frac{1}{2}} = \qquad \ldots 5)$$

$$\text{with } T = \frac{2\pi}{\omega}\ ;\ T_1 = \frac{2\pi}{\omega_1} \qquad \ldots 6)$$

By definition

$$|x_s| = \frac{|F'|}{K'} \qquad \ldots 7)$$

where xs is the statically derived displacement.

Hence the one degree of freedom d.a.f. γ may be defined by

$$|x| = |x_s|\,\gamma \qquad \ldots 8)$$

This function γ is plotted in figures 3a and 3b for T_1 = 3 and 6 secs.

The fundamental sway mode of a steel jacket in water corresponds closely to the global response shape under long wave loading. This global response shape is close to the fundamental sway mode and gives rise to "frame action" loads in structural members.

For the storm condition where harmonic loads with periods between 14 and 19 seconds are applied to the structure this approach may be used 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 or when the structural fundamental period is close to periods of appreciable wave energy within the storm sea state.

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

2.2 Spectral Dynamic Amplification Factors

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

$$\sigma_d^2 = \int_0^\infty S_{\eta\eta}(\omega)\left|T_{dyn}(\omega)\right|^2 d\omega \qquad \ldots 9)$$

where T dyn is the dynamic transfer function for the chosen degree of freedom for the structure considered and is calculated 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:-

$$S_{\eta\eta}(\omega) = \frac{H_s^2 T_3}{4\pi}\left(\frac{T_3\omega}{2\pi}\right)^{-5} \exp\left[-\frac{1}{\pi}\left(\frac{\omega T_3}{2\pi}\right)^{-4}\right] \quad \ldots 10)$$

where T_3 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,

$$T_3 = T \times 0.81 \quad \ldots 11)$$
$$\& \; H_s = H/1.8 \quad \ldots 12)$$

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. and the static response, then:-

$$T_{dyn}(\omega) = T_{stat}(\omega)\,\gamma(\omega, \omega_n) \quad \ldots 13)$$

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

$$\sigma_s^2 = \int_0^\infty S_{\eta\eta}(\omega)\,|T_{stat}(\omega)|^2\,d\omega \quad \ldots 14)$$

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

$$\gamma_{spect}^2 = \frac{\sigma_d^2}{\sigma_s^2} = \frac{\int_0^\infty S_{\eta\eta}(\omega)\,|T_{stat}(\omega)|^2 \gamma(\omega,\omega_n)\,d\omega}{\int_0^\infty S_{\eta\eta}(\omega)\,|T_{stat}(\omega)|^2\,d\omega} \quad \ldots 15)$$

Evaluation of this expression requires detailed knowledge of the form of the transfer function $T_{stat}(\omega)$.

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 $S_{\eta\eta}(\omega)$.

Cancellation of this function now becomes possible and:

$$\gamma_{spect}^2 = \frac{\int_0^\infty S_{\eta\eta}(\omega)\,\gamma^2(\omega,\omega_n)\,d\omega}{\int_0^\infty S_{\eta\eta}(\omega)\,d\omega} \quad \ldots 16)$$

This is now a dynamic amplification factor which takes due account of the spread of frequencies in the sea state 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.

This spectral d.a.f is compared with the conventional d.a.f. for a range of sea states in Figure 3a 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 $T_3/0.81$. 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 situation for a structure with natural period 6 seconds is somewhat worse (figure 3b). 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.

2.3 Relationship with Multi Degree of Freedom Systems

The response of Multi degree of freedom systems such as the computer models shown in Figures 1a and 1b may be represented by the matrix equations:

$$\underline{M}\,\ddot{\underline{X}} + \underline{C}\,\dot{\underline{X}} + \underline{K}\,\underline{X} = \underline{F} \quad \ldots 17)$$

Where $\underline{M}$, $\underline{C}$ and $\underline{K}$ are the (nxn) mass, damping and stiffness matrices for the structure, $\underline{X}$ is the nodal displacement vector and $\underline{F}$ is the load vector.

Assuming that $\underline{C}$ is a linear combination of $\underline{M}$ and $\underline{K}$ these matrix equations may be de-coupled into n linear equations by solution of the corresponding eigen-value problem.

$$(-\omega_i^2 \underline{M} + \underline{K})\,\underline{\phi}_i = \underline{O} \quad \ldots 18)$$

where ω_i (i = 1 ...... n) are the eigen-frequencies (lowest first) and ϕ_i are the eigen-vectors or mode shapes for each of the n solutions of 18.

If the transformation matrix $\underline{\underline{\phi}}$ consisting of the n eigen-vectors as columns is constructed, pre-multiplication of 17 by the transpose $\underline{\underline{\phi}}^T$ results in the n uncoupled equations

$$m_i \ddot{x}_i + c_i \dot{x}_i + k_i x_i = f_i \quad ;(i = 1 \ldots n) \quad \ldots 19)$$

and we may define

$$\xi_i = \frac{c_i}{2\sqrt{m_i k_i}} \quad \ldots 20)$$

with mi, ci and ki being the generalised mass, damping and stiffnesses associated with the generalised coordinates xi.

fi is the generalised force given by the elements of $\underline{f}$ defined by:-

$$\underline{\underline{\phi}}^T \underline{F} = \underline{f} \quad \ldots 21)$$

and xi is the generalised displacement given by the elements of $\underline{x}$ defined by:-

$$\underline{X} = \underline{\underline{\phi}}\,\underline{x} \quad \text{or} \quad \underline{x} = \underline{\underline{\phi}}^T \underline{X} \quad \ldots 22)$$

The solution for xi under harmonic loading at angular frequency ω_i is given by:

$$x_i = \frac{f_i}{k_i}\left\{\left(1 - \left(\frac{\omega}{\omega_i}\right)^2\right)^2 + \left(2\xi\frac{\omega}{\omega_i}\right)^2\right\}^{-\frac{1}{2}} = \frac{f_i}{k_i}\gamma_i \quad \ldots 23)$$

where γ_i may be interpreted as the dynamic amplification factor associated with the ith mode.

fi/ki may be interpreted as the generalised static response.

The correspondence between equations 4 and 23 is now clear and the following points may be noted:-

1. For the first mode (i=1) $\gamma_i \approx \gamma$ the conventional one degree of freedom d.a.f. This is inherently larger than γ_i ; i = 2.... n for wave frequencies ω below ω_i.

2. The first mode shape ϕ_1 corresponds to the total deformed shape under wave loading for jackets such as those shown in figures 1a and 1b.

3. The wave force vector $\underline{F}$ has a vertical distribution which closely mirrors the first mode shape $\underline{\phi}1$ due to the attenuation of water particle velocities and accelerations with depth. As a result f1 the first generalised force will be the largest element of $\underline{f}$ (depending on wave direction correspondence with $\underline{\phi}1$ or $\underline{\phi}2$) as can be seen from equation 21 and the fact that the first row of $\underline{\phi}^T$ is $\underline{\phi}_1$. For the same reason, the first generalised coordinate x1 may be associated with the horizontal displacement of the topside centre of gravity as assumed in Section 2.1

For the above reasons the global dynamic amplification under harmonic loading is often determined by the one degree of freedom dynamic amplification factor associated with the fundamental sway mode in the appropriate direction γ. This amplification is applicable to stress ranges induced by global frame action deformations. Local vibration effects should be dealt with using the appropriate local modal frequency.

In order to quantify the errors associated with these assumptions, a direct comparison study was performed by direct calculation of dynamic and static stresses for the realistic structures shown in Figures 1a and 1b.

3.0 COMPARISON STUDY

3.1 Description

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

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 dynamic response to the waves shown in Table 2 was then calculated by modal 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.

3.2 Results

The axial forces in the legs were found to be 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 general high level of load at the base of the structure indicates the importance of correct dynamic amplification at these locations.

Figure 4 shows 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 for both jackets 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.

Figure 5 shows 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.

Figure 6 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. Waves near resonance indicate the unconservatism in the conventional d.a.f.

3.3 Frame Action and Dynamic Amplification

In Section 2.3 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. Usually the d.a.f. corresponding to the sway mode period is conservatively applied to the total stresses/displacements.

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:-

$$S_d = \gamma_1 S_F + \gamma_n S_L < \gamma_1 (S_F + S_L) = \gamma_1 S \quad \ldots 24)$$

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

where γ_1 is the d.a.f. for the fundamental 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 4 , 5 and 6 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 figure 7 were determined with and without local hydrodynamic loading. Frame action loading = total - local loading.

As expected, the largest contribution of local wave loading occurs in horizontal members near the splash zone. Figure 8 shows the variation of (vertical) shear force for such a member at -15m for jacket 2 for waves travelling perpendicular to the member axis with heights of 1, 3, 6, 9 and 12m at 9 second period.

In this case direct loading contributes 80% of the total shear. The axial load due to frame action though small still dominates for this member and wave direction. The frame action component of force varies linearly with wave height at this period. The local wave induced shear exhibits a small degree of non-linearity as drag becomes dominant for the higher waves.

For waves travelling parallel to the member axis the variation of axial force with height shows that the frame action load dominates. The same qualitative behaviour was observed for axial load in the other members considered. Both components vary almost linearly with wave height.

The results indicate that a combination of local and global d.a.f's is appropriate for degrees of freedom associated with shear at the ends of braces near the splash zone for waves perpendicular to the member axis. Our results indicate that axial forces may be safely amplified using a d.a.f based on the sway natural period.

4.0 FATIGUE AND DYNAMIC AMPLIFICATION

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.

4.1 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.[3]

   Therefore $N \propto S^{-3} \sim H^{-3\alpha}$ ... 25)
3. Damage is the ratio of number of occurrences to the number of cycles to failure (N) at stress range S from the SN curve [3].

$$D = \frac{n}{N} \sim nH^{3\alpha} = nS^3 \qquad \text{... 26)}$$

This process is illustrated in figures 10a and 10b. 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, in the past taken to be 1.8. we use a value between 1 and 1.4. Our results indicate that a value of near unity gives the best fit. Figure 9 shows 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.

4.2 Dynamic Amplification Factors for Fatigue

Dynamic amplification has two major effects:-

1. The damage distribution shown in figure 10a is biased towards the sway period, 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:-

$$S_d = S\gamma \qquad \text{... 27)}$$

with γ being evaluated at each wave period,

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

$$D_d = \frac{n}{N} \sim n(\gamma S)^3 = nS^3\gamma^3 = D\gamma^3 \qquad \text{... 28)}$$

hence point by point

$$\gamma = \left(\frac{D_d}{D}\right)^{1/3} \qquad \text{... 29)}$$

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

The cumulative damage curve may now be constructed and is displaced towards lower wave heights and periods (see figure 10b).

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:

di - di-1
and with dynamic amplification
d'i - d'i-1

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

$$\frac{\text{Damage for range i (dynamic)}}{\text{Damage for range i (static)}} = \frac{d'_i - d'_{i-1}}{d_i - d_{i-1}} \quad \text{... 30)}$$

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:-

$$\gamma_i^3 = \frac{d'_i - d'_{i-1}}{d_i - d_{i-i}} \quad \text{... 31)}$$

These 'fatigue' dynamic amplification factors 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 sway natural periods as the conventional approach will vastly overestimate the damage associated with representative waves near sway resonance.

5.0 CONCLUSIONS

1. A spectral dynamic amplification factor has been derived which takes into account the frequency spread of sea states associated with a design wave. Results indicate that the conventional d.a.f. will be unconservative for design waves above structural sway resonance.

2. Consideration of frame action loading indicates that for bracing members near the splash zone a d.a.f. based on the local natural period in combination with a global sway d.a.f. is more appropriate for shear loads especially for waves travelling perpendicular to the member. The global d.a.f. is suitable for amplification of axial loads.

3. Investigations of the relationship between stress range S and wave height H, indicate that:

   S H with less than the 1.8 normally assumed. A value of between 1 and 1.4 has also been found on other major jacket structures designed by John Brown. The value to be used in design will depend on the assumed wave slope and hence the relationship between H and wave period.

4. A fatigue d.a.f. has been postulated which is based on damage amplification. Use of this d.a.f. will affect the choice of the representative wave and avoid overestimation of fatigue damage due to small, short waves.

6.0 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 Butterworths, 1980.

3. "Offshore Installations: Guidance on design and constructions", Department of Energy, HMSO ISBN 0 11 701118 5, Third Edition, 1984.

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.

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	x	x		x		x		x	x			x	
3					x			x							
6							x	x							
8								x							
9								x							
10									x						
12								x							
14											x				
18												x			
20													x		
30														x	

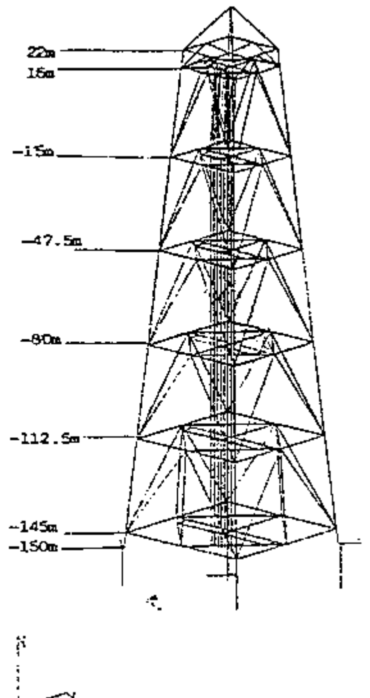

Figure 1a Jacket 1 150m Water Depth

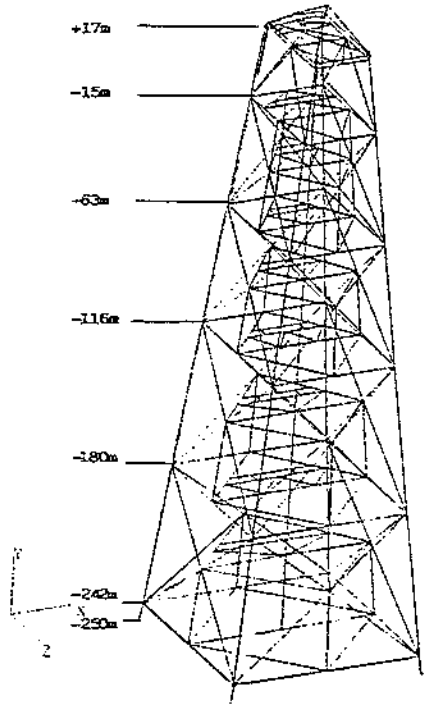

Figure 1b Jacket 2 250m Water Depth

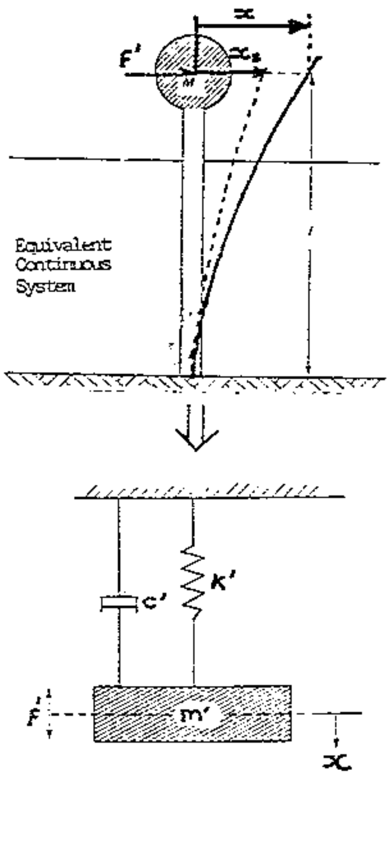

Figure 2 One Degree of Freedom Idealisation

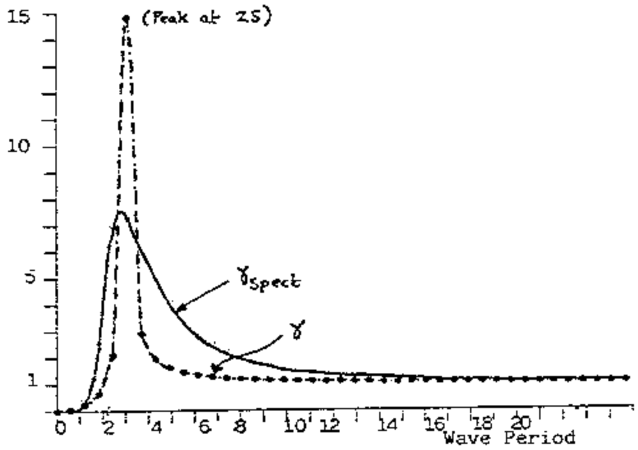

Figure 3a Dynamic amplification factors (T_1 =3 secs)

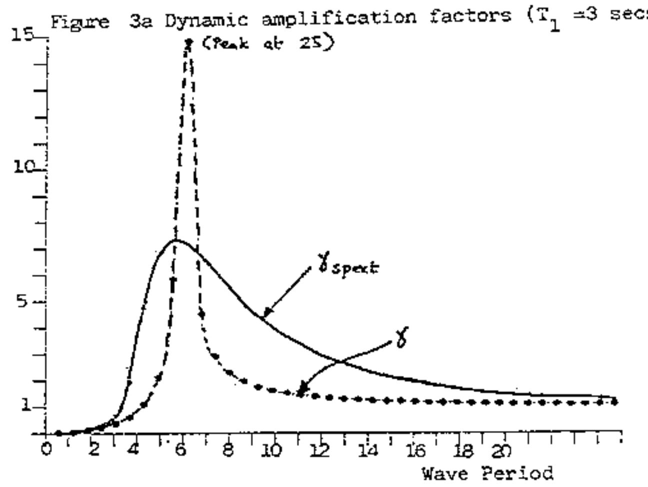

Figure 3b Dynamic amplification factors (T_1 = 6 secs)

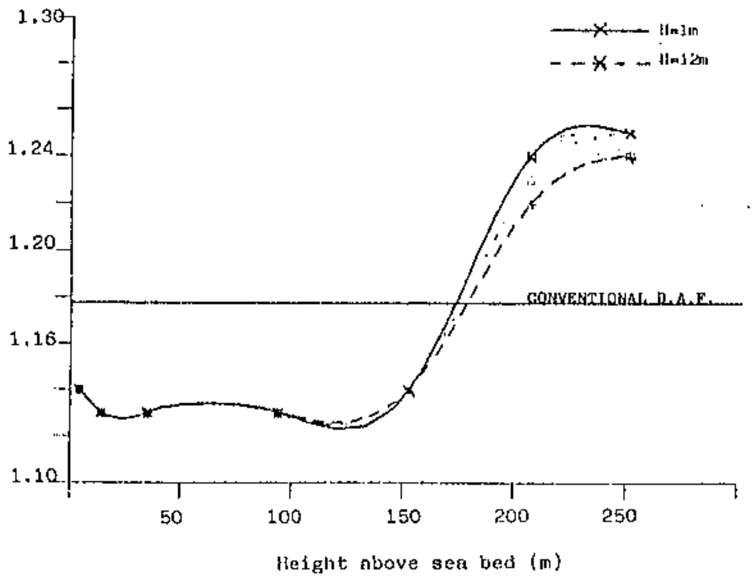

Figure 4 Dynamic amplification factor (9 second wave) from leg nodes (Jacket 2)

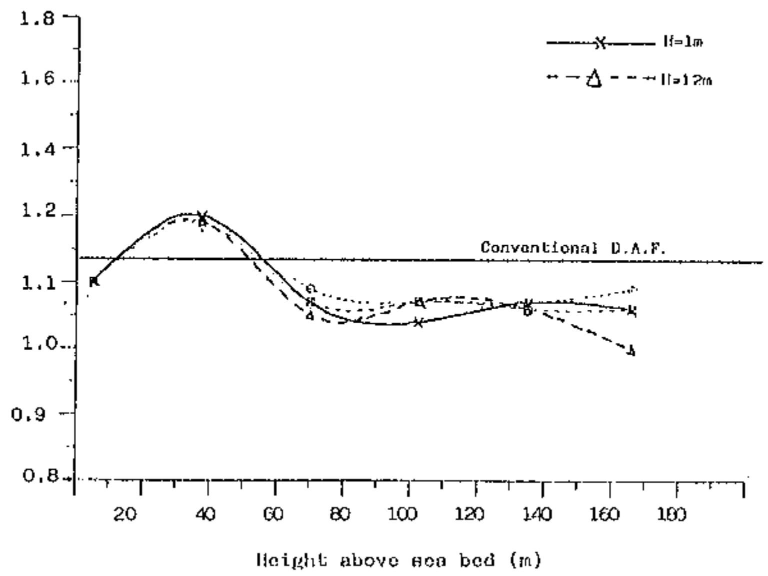

Figure 5 Dynamic amplification factor (9 second wave) from brace axial forces Jacket 1

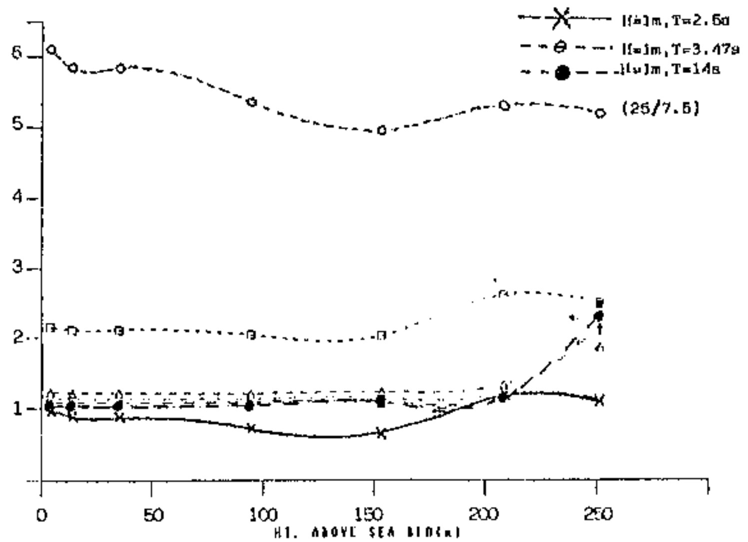

Figure 6 Dynamic Amplification factor Vs. Period (Jacket 2)

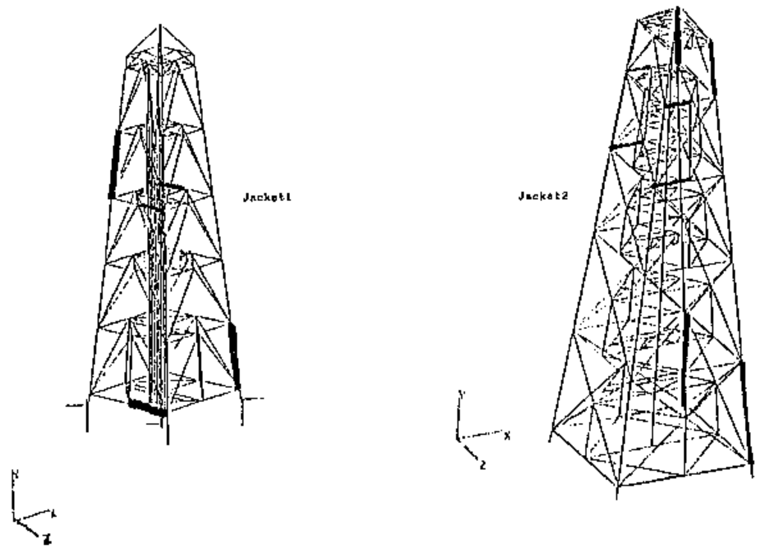

Figure 7 Frame action study - Unloaded Members

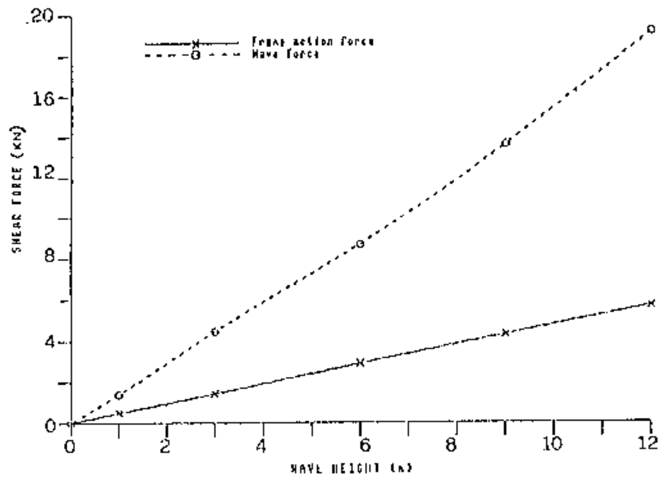

Figure 8 Frame action and Local Loading Vs wave height (period = 9 secs)

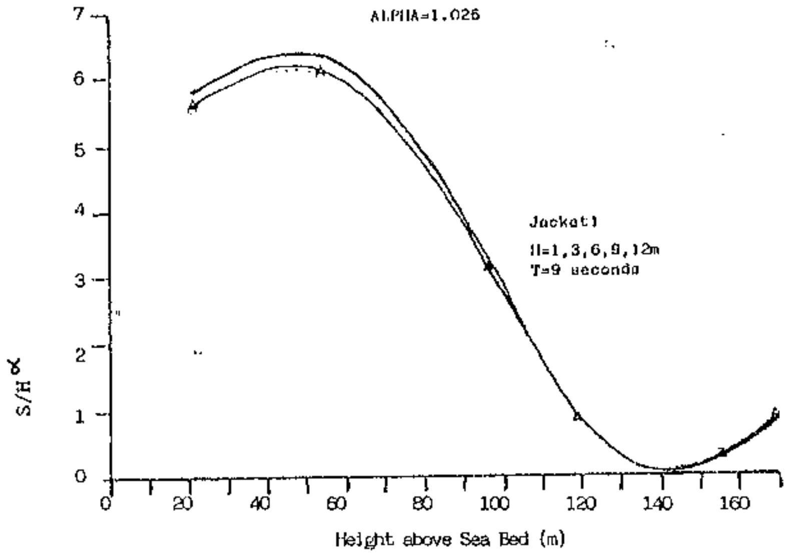

Figure 9 Stress Vs Wave Height

Table 4 Fatigue d.a.f.'s

Static Wave Height	Dynamic Wave Height (m)	Period (s)	No of Cycles	Fatigue d.a.f.	Conventional d.a.f.
1.93	1.84	4.34	1.301×10^{8}	1.86	2.01
4.63	4.55	6.85	7.851×10^{6}	1.26	1.25
6.15	6.09	7.99	3.219×10^{6}	1.17	1.17
7.99	7.93	9.01	1.437×10^{6}	1.13	1.13
11.17	11.14	10.70	4.975×10^{5}	1.08	1.08

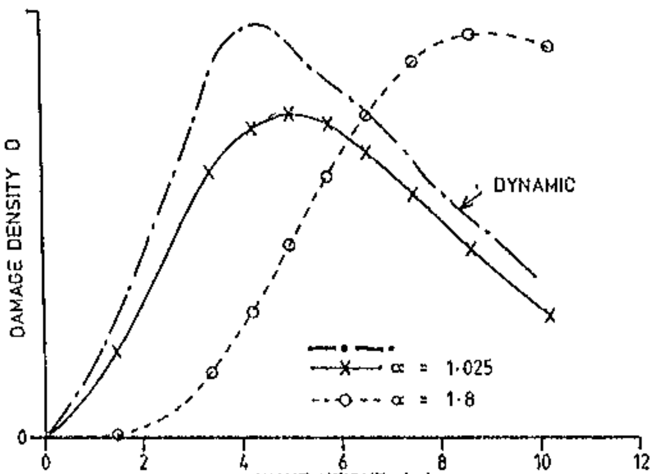

Fig. 10a
TYPICAL DAMAGE DISTRIBUTION AGAINST WAVE HEIGHT

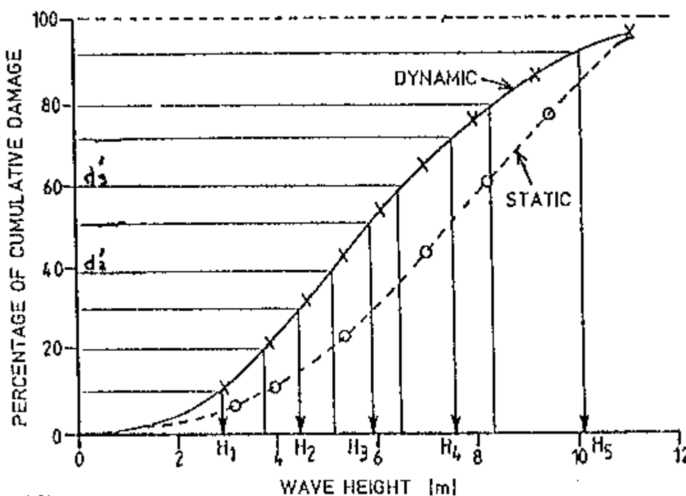

Fig. 10b
CUMULATIVE DAMAGE DISTRIBUTION / CHOICE OF FATIGUE WAVES