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Nonlinear Water-Structure Interaction of Fixed Offshore Platform in Extreme Storm

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ABSTRACT

In this paper a simplified method for dynamic response of jacket-type offshore structures to extreme environmental load is investigated using existing experience and the procedures available within the industry. Since wave loads on offshore platforms varied with time they produce dynamic effects on structures. An accurate and highly efficient procedure for the random vibration computation of complex engineering structures, known as pseudo excitation method (PEM), has been developed since the 1980s (Hongyan et al, 2004). To illustrate the aforementioned methodology, a jacket type offshore platform in the Persian Gulf has been selected.

KEY WORDS: dynamic-response; jacket-type; offshore-platform; pseudo-excitation; time-history.

INTRODUCTION

A complete model including topsides, jacket, piles and the surrounding soil is considered using SACS software (refer to Fig. 1). For Platform description and environmental data refer to Table 8 in the Appendix. The non-linear force deformation behavior of the pile soil system is modeled to account for finite deflections of the pile (the P-delta effect). Dynamic characteristics (mode shapes and frequencies) of the structure are generated with reduced structural stiffness matrix and consistent mass approach. A random sea state (surface elevation) can be represented by a Gaussian process; however wave loading is nonlinear due to the drag term in the Morison wave load equation and inundation effects. Therefore fixed offshore platform response to random wave loading will no longer be Gaussian in nature (Greevs et al, 1996). A frequency domain-spectral analysis technique is clearly able to reflect the random nature of the wave loading via the combination of the structural transfer functions with a wave spectrum. However it is not able to directly capture the nonlinear wave loading behavior. The use of frequency domain-spectral analysis techniques requires the response behavior of the structure to be linearized; therefore random dynamic calculations have been performed through time domain simulations to account for nonlinearities in non-Gaussian process (Soding et al, 1990). The link between the random dynamic response and the quasi-static design event forces is achieved by making use of global dynamic amplification factors (Vugts et al, 1998). An additional static load (Separate inertia load set) is combined with the quasi-static extreme storm environmental loads to reflect the inclusion of dynamic effects. Inertia load set which represent the distribution of mass inertia forces over the height of the structure are applied at the main plan levels in proportion to the mass distribution in order to achieve target dynamic level shear and overturning moment values. The suitability of a method as a practical assessment tool is investigated as a relevant response process.

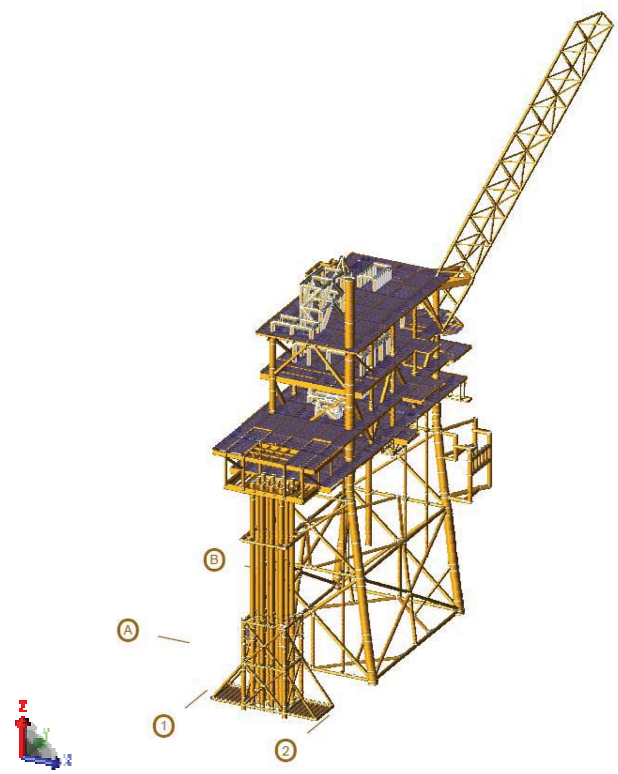

Fig. 1 General view of SACS structural model

DYNAMIC CHARACTERISTICS

Since the dynamic module in SACS uses the linear theory (i.e. modal superposition), linearized foundation super elements are automatically created at each pile-head by the PSI program using the maximum pile stiffness for each pile group specified in the input files. Different load cases are used to calculated stiffness in each orthogonal direction as well as the diagonal direction. When dealing with very large structures (particularly lattice structures) the technique of *mass condensation* may be used (Guyan et al, 1965). A set of master (retained) degrees of freedom are selected at each horizontal elevation on the main legs to extract the Eigen values (periods) and Eigen vectors (mode shapes), which includes all stiffness and mass properties related to the reduced degrees of freedom. After the modes are extracted using the master degrees of freedom, they are expanded to include the full 6 degrees of freedom for all joints in the structure. A consistent mass approach is considered since it is more desirable for structures immersed in the fluid. The added mass is generated automatically by SACS and depends on the size, orientation and proximity of the member to the free surface (Asgarian et al, 2004). Entrapped mass is calculated for members designated as flooded in the model file. Hydrodynamic effects of marine growth are included in the program to account for the density and effective diameter due to marine growth. The normalized mode shapes for the first two lateral mode shapes of vibration in each direction is shown below:

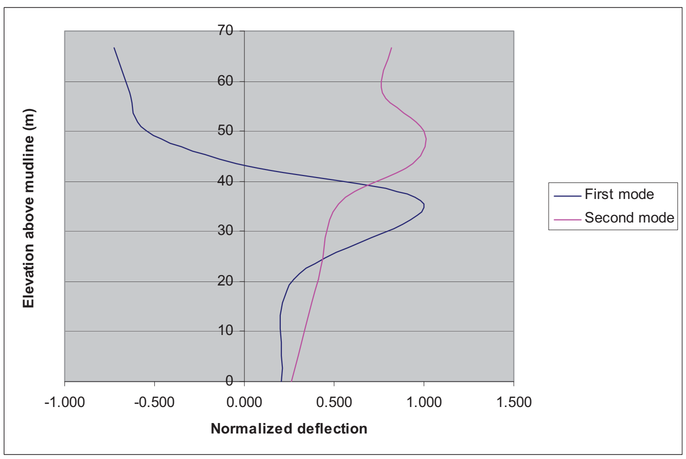

Fig. 2 mode shape in X-direction

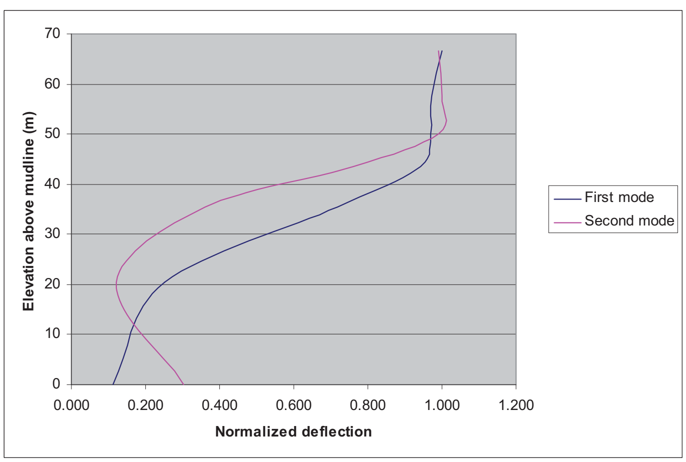

Fig. 3 mode shape in Y-direction

The associated lumped mass distribution for these mode shapes is calculated using the results from the dynamic analysis as below:

Table 1. Load distribution matrix

ELEVATION (m)	Self Weight + plates(KN)	Lumped mass (KN)	Entrapped Fluid (KN)	Fluid Added Mass(KN)	Total (kN)
-29.525	3385.02	2562.82	651.37	697.74	7297
-9	9814.18	1240.39	1950.41	2015.23	15020
6	1322.36	385.38	43.35	136.30	1887
13.9	4785.62	13016.62	0	0.00	17802
20.5	4348.46	14396.99	0	0.00	18745
28.1	3485.32	5315.20	0	0.00	8801
37.1	2479.66	6049.81	0	0.00	8529
	29621	42967	2645	2849	78082

RANDOM TIME DOMAIN ANALYSIS PROCEDURE

Random wave analysis is required for flexible structures which respond dynamically during extreme storm conditions. The wave response module of SACS is used in random wave mode to perform the dynamic response of a structure using the full three-dimensional SACS model and dynamic characteristics (mode shape and mass). Twenty modes are considered in the analysis. In the random wave procedure Pierson – Moskowitz spectrum is specified with time duration of 1200 sec. The spectrum is analyzed and broken into Airy wave components. Only wave components with periods that are divisible into the analysis time duration are considered as possible components. The period of a possible wave component, n, is determined from the following equation:

$$T_n = T_D/n \quad n = 1, 2, 3, ..., n_{max} \tag{1}$$

Where T_n is the period, T_D is the time duration and n_{max} is the last significant component which is 30 in this analysis. Here for an analysis time duration of 1200 seconds the period of first possible component is 1200 seconds, the period of the second possible component is 600 seconds, the third is 400 seconds.

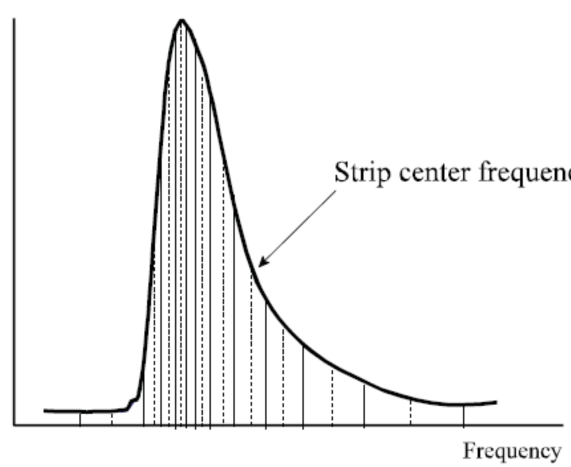

Fig. 4 input spectrum

The input spectrum is divided into strips (refer to Fig. 4), with each strip having a center frequency corresponding to the frequency of one of the possible wave components. The strips are then lumped together so that each lumped strip contains at least the minimum portion of the spectrum allowed (here the default value of 1% is considered).

The program reproduces the spectrum by linear addition of Airy waves. The results shows that there is no discrepancy between the theoretical and calculated wave spectra, since the time duration of the surface time history was long enough to represent the statistical properties sufficiently, refer to Fig 5.

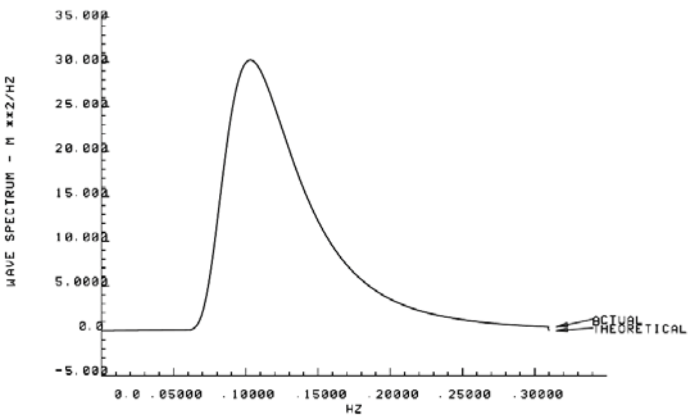

Fig. 5 Wave spectrum

For random wave analysis, random linear waves with modified crest kinematics are generated based on wave spectra. The reference wave is selected to wet the maximum height likely to occur during the random wave. Linear combinations of linear Airy waves are used to simulate the wave surface profile. The change in the position of the structure due to its own motion is considered. The displacements of the structure are usually very small as compared to the lengths of the significant wave components, therefore the wave kinematics from the first iteration are used for all subsequent iterations assuming that there is no significant loss of accuracy. The effects of a steady horizontal current are included in the random wave analysis. A set of 20 user input seeds is used to generate numerous random surfaces of the random wave (Fish, 1980) and responses calculated for each profile at 0.5 second intervals. For design applications, only extreme waves are of interest, therefore from each of the twenty simulations the largest five peak responses of static and dynamic base shear forces as well as static and dynamic overturning moments were picked from independent waves to make a total of 100 data points for each response variable (refer to table 6 in the Appendix). The DAFs (Dynamic Amplification Factor) for BS (base shear) and OTM (overturning moment) are derived from dividing the ordered sequence of dynamic responses by the corresponding ordered sequence of the static responses. The results are shown in Fig. 6 and Fig. 7 for two orthogonal directions.

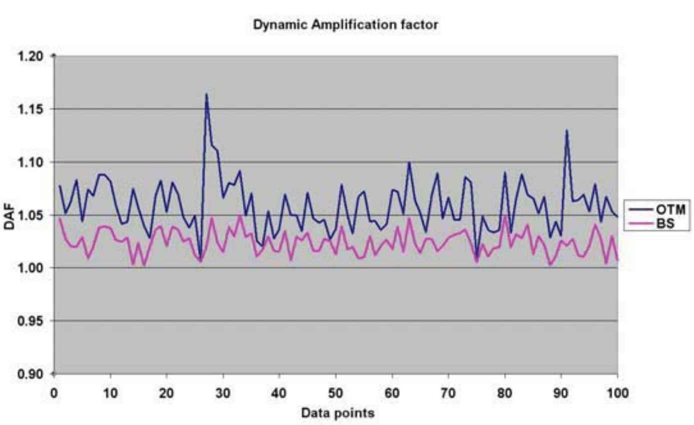

Fig. 6 wave in the x-direction

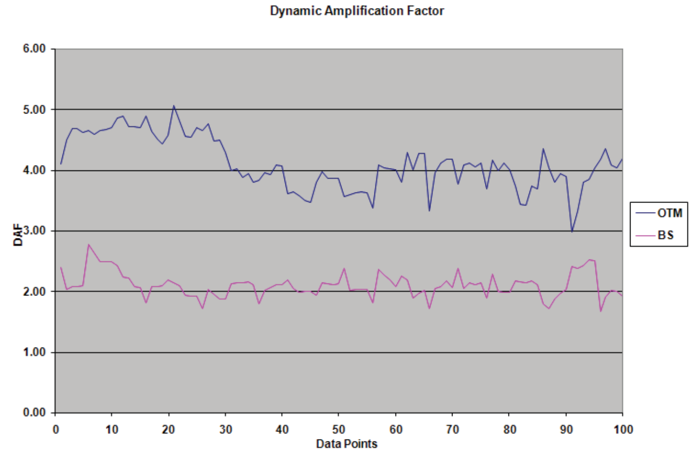

Fig. 7 wave in the y-direction.

The results show that the DAF is higher in Y direction compared to X direction. Since the jacket is designed for float-over installation there is no X-bracing provided in the top bay in Y direction and the structure is more flexible and dynamically sensitive for waves approaching in this direction. The median values of the DAFs from these plots are given in Table 2.

Table 2. Dynamic Amplification Factor

DAFs	X-dir	Y-dir
Base shear	1.02	2.07
Overturning moment	1.05	4.02

Further calculation is carried out for base shear DAFs using conventional method as a response to simple harmonic loading (Hallam et al, 1977) which is followed:

$$T_{fx} := 11.3sec \qquad T_{nx} := 1.78sec \qquad \xi := .02 \qquad \Omega_x := \frac{T_{nx}}{T_{fx}} \tag{2}$$

$$DAF_x := \frac{1}{\sqrt{\left(1-\Omega_x^{\,2}\right)^2 + \left(2\cdot\xi\cdot\Omega_x\right)^2}} \qquad DAF_x = 1.025 \tag{3}$$

$$T_{fy} := 8.5sec \qquad T_{ny} := 3.01sec \qquad \Omega_y := \frac{T_{ny}}{T_{fy}} \tag{4}$$

$$DAF_y := \frac{1}{\sqrt{\left(1-\Omega_y^{\,2}\right)^2 + \left(2\cdot\xi\cdot\Omega_y\right)^2}} \qquad DAF_y = 1.143 \tag{5}$$

T_f stands for forcing period of the wave and T_n stands for natural period of the structure (refer to table 7). ζ stands for damping ratio.

A comparison of the DAFs from random time domain analysis and conventional methods shows that the DAFs in x-direction are the same; however there is a huge difference for DAF in y-direction. The DAF in y direction derived from random time domain analysis is about two times bigger than the one derived from the conventional method. As a conclusion if there is any discontinuity in jacket braces which results in higher natural periods, the conventional methods for DAF calculation is no more reliable. The extent of dynamic response relates to the structure's fundamental modes and frequency content of the applied loading. Design and analysis procedures for structures which respond dynamically must account for the random nature of the excitation by random time domain simulation or frequency domain spectral analysis.

INERTIAL SHEAR FORCES

For a low loading frequency the total response can be represented by the contribution of the lower vibration modes and a pseudo-static correction, to take into account the contributions of the neglected higher modes (Lima et al, 1985). To determine the extreme response of the structure for the purpose of design against strength and stability a pseudo-dynamic design wave procedure is used. The inertia load set is an additional static load which, when combined with the quasi-static extreme storm load, results in global levels of response which reflect the inclusion of dynamic effects (Greeves et al, 1995). For random excitation the influence of dynamics on global loading can be expressed by a generalized form of the Dynamic Amplification Factor (DAF) through the random analysis techniques. Determination of the inertial load set is achieved by making use of global responses (Vugts et al, 1998). This method modifies the static shear forces by dynamic amplification factors. Inertia shear forces are proportional to the fundamental bending modes of the structure for the end-on and broadside wave directions at the main plan levels. This simplified approach incorporates the essential features of the platform dynamic response and eliminates the difficulties associated with use of the individual member forces from a random wave analysis to suit with existing codes of practice. Random wave analysis in time domain is used to determine the appropriate global (level shear and moment) dynamic amplification factors. As a result the random time domain analyses and the prediction of the associated DAFs is limited to the global response only, which is technically feasible and economical. The appropriate inertia load set is the summation of mass inertia forces of motion resulting from the first two fundamental modes:

So that:

$$V1 = M.\Phi 1 \quad (6)$$

$$V2 = M.\Phi 2 \quad (7)$$

Where:

Vi & Mi = vector of lateral modal forces and moments in mode i, refer to Table 3 & 4.

M = system mass matrix

Φi = mode shape for i^{th} mode (refer to fig 2 & 3)

Assuming full participation of each mode the resulting inertial base shear and overturning moments for mode i, are obtained by summing the contributions from each level as follows:

Modal Shear in mode i,

$$Vi = \sum_{j=1,n} \Phi_{ij}\, m_j \quad (8)$$

Modal Moment in mode i,

$$Mi = \sum_{j=1,n} \Phi_{ij}\, h_j m_j \quad (9)$$

where:

j = plan level subscript

n = total number of lumped masses

i = mode number

m_j = lumped mass at level j

h_j = height above base to level j

Φ_{ij} = mode shape amplitude for mode i at level j

For time domain simulations the inertial response is obtained by direct subtraction of the two random dynamic and quasi-static responses. The dynamic components of the base shear and overturning moment are then developed from the sum of the modal contributions. Inertial responses in each mode are proportional to the product of the mass and mode shapes. The contributions from each mode are a function of the modal participation factors α_i which depends upon the characteristics of the loading and the mode shape.

The proportionality (or participation) coefficients (α_i) for each mode are determined by solving the following equations, assuming that two modes are relevant (refer to Table 5).

$$\text{Inertial Shear} = \alpha 1 \sum_{j=1,n} \Phi_{1j}.m_j + \alpha 2 \sum_{j=1,n} \Phi_{2j}.m_j \quad (10)$$

$$\text{Inertial Moment} = \alpha 1 \sum_{j=1,n} \Phi_{1j}.h_j.m_j + \alpha 2 \sum_{j=1,n} \Phi_{2j}.h_j.m_j \quad (11)$$

The inertia load sets for each mode and for each principal direction is calculated by multiplying the modal load sets developed above using the appropriate modal participation (α) values (refer to Fig. 8 & 9). The inertia load set is applied to the structure as a static extra load set in addition to all of the other environmental and self weight loads. The inertia load set is combined with a regular wave deterministic analysis to design the structure in the same manner as for conventional structures. The above procedure is applied to both principal wave approach directions. The above relationships match the required base shear and overturning moment using two proportionality constants α_1, α_2, and two modes. Level shears and moments at any other location can also be matched in a similar manner.

If the dynamic response components are to be matched at more than two levels (say n) then n modes and n proportionality constants must be used resulting in a set of n equations similar to the above. However in this example only 2 modes and 2 proportionality constants have been used based on the assumption that the dynamic response contributions are dominated by the first two fundamental modes (refer to the Table 2). As the influence of higher modes increases the above approximation becomes more inaccurate.

Table 2. Mass Participation Factors

MODE	X	Y	Z
1	0.0000006	0.8204587	0.0000100
2	0.7685387	0.0000250	0.0000058
3	0.0985639	0.0000992	0.0000014
4	0.0000059	0.0671181	0.0000014
5	0.0659842	0.0000139	0.0000013
6	0.0001300	0.0000006	0.0000000
7	0.0438895	0.0026830	0.0017154
8	0.0010757	0.0963786	0.0002689
9	0.0007284	0.0031202	0.0013132
10	0.0047347	0.0001783	0.0181825
11	0.0018160	0.0000054	0.0000830
12	0.0000030	0.0000336	0.0000121
13	0.0003991	0.0077855	0.0000080
14	0.0054696	0.0002324	0.0042012
15	0.0065685	0.0000463	0.0000616
16	0.0000002	0.0000098	0.0000154
17	0.0001224	0.0000106	0.0355004
18	0.0000349	0.0000057	0.7826106
19	0.0000523	0.0000000	0.0038140
20	0.0000160	0.0002289	0.0006175

Table 3. Modal load in x-direction

	Mass	Shear		Moment	
		V1	V2	M1	M2
EL (m)	Tonnes	Tonnes	Tonnes	Tonnes m	Tonnes m
0.00	743.83	152.15	196.54	0.00	0.00
20.52	1531.09	417.57	627.76	8568.54	12881.67
35.52	192.35	192.35	101.41	6832.44	3602.06
43.42	1814.68	-82.49	1700.73	-3581.52	73845.81
50.02	1910.81	-1042.26	1910.81	-52133.72	95578.48
57.62	897.15	-570.91	691.33	-32895.89	39834.72
66.62	869.42	-632.30	709.16	-42124.14	47243.98

Table 4. Modal load in y-direction

	Mass	Shear		Moment	
		V1	V2	M1	M2
EL (m)	Tonnes	Tonnes	Tonnes	Tonnes m	Tonnes m
0.00	743.83	82.91	224.76	0.00	0.00
20.52	1531.09	385.78	183.58	7916.15	3767.14
35.52	192.35	138.23	69.19	4909.81	2457.71
43.42	1814.68	1712.83	1379.50	74371.28	59898.07
50.02	1910.81	1855.31	1892.48	92802.65	94661.66
57.62	897.15	872.85	897.15	50293.67	51693.54
66.62	869.42	869.42	861.08	57920.69	57365.10

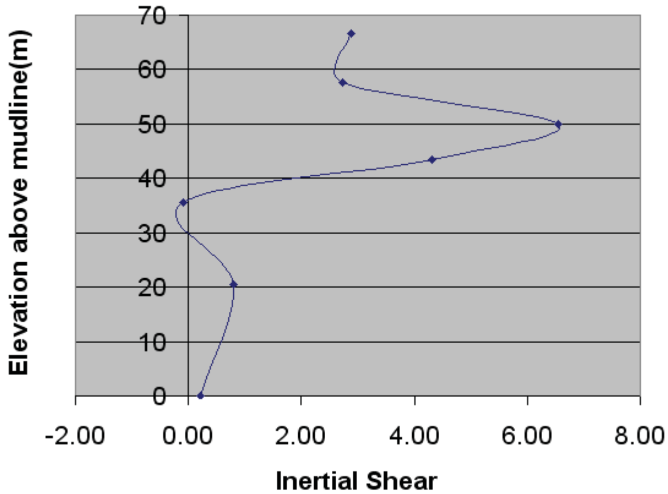

Fig. 8 Inertial shear in x-direction

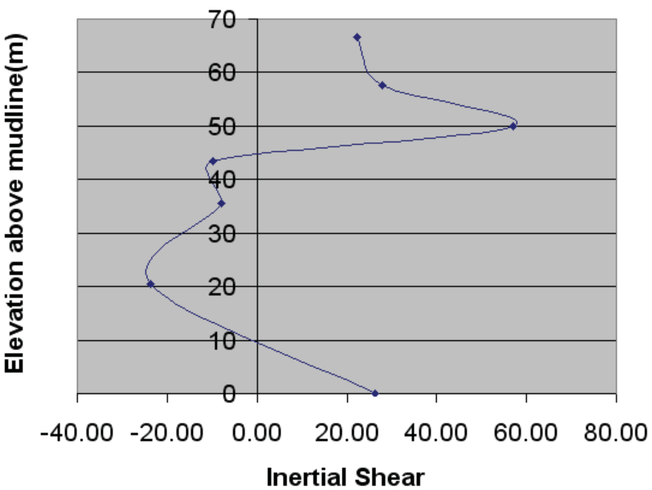

Fig. 9 Inertial shear in y-direction

Table 5. Modal Participation factors

Participation coefficients	x-direction	y-direction
α_1	-0.001792519	-0.14229167
α_2	0.002451498	0.169660792

CONCLUSIONS

Fixed offshore platform response to random wave loading is not Gaussian in nature. A frequency domain-spectral analysis technique is able to reflect the random nature of the wave loading via the combination of the structural transfer functions with a wave spectrum. The use of frequency domain-spectral analysis techniques requires the response behavior of the structure to be linearized, thus it is not able to directly capture the nonlinear wave loading behavior. Time domain simulations in random waves is able to include the response non-linearities due to drag force and free surface variation (Bar-Avi, 1959). Therefore random dynamic calculations have been performed through time domain simulations to account for nonlinearities in non-Gaussian process. Pseudo Dynamic Analysis is a simplified dynamic structural procedure suitable for the conventional bottom supported structures under the influence of environmental loading and self weight forces. Given the potential impact of non-linear components in the extreme storm condition as illustrated in this paper, it is now accepted that models which reflect these features explicitly are required.

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APPENDIX

Table 6. Global peak response data points in X direction*

Data points	Seed No.	Dynamic				Static				DAF	
		TIME (SECS)	OTM (kN.m)	TIME (SECS)	BS (kN)	TIME (SECS)	OTM (kN.m)	TIME (SECS)	BS (kN)	OTM	BS
1		641	75070	641	3925	603	69685	641	3751	1.08	1.05
2		603	72790	603	3851	641	69222	603	3749	1.05	1.03
3	1	604	66535	641	3775	604	62625	641	3701	1.06	1.02
4		899	64968	711	3511	899	59981	711	3443	1.08	1.02
5		411	62601	411	3443	711	59977	411	3346	1.04	1.03
6		607	77899	607	4124	607	72500	607	4089	1.07	1.01
7		414	66571	608	3864	414	62341	608	3790	1.07	1.02
8	2	716	65582	414	3637	583	60273	414	3502	1.09	1.04
9		606	65466	716	3631	606	60167	716	3495	1.09	1.04
10		103	64521	606	3564	645	59667	606	3433	1.08	1.04
11		642	75344	641	3946	642	71098	641	3846	1.06	1.03
12		603	73946	603	3937	603	70987	603	3842	1.04	1.02
13	3	641	73182	642	3935	641	70147	642	3825	1.04	1.03
14		604	69985	712	3611	604	65124	712	3600	1.07	1.00
15		103	65374	713	3598	712	61854	713	3514	1.06	1.02
16		607	74860	607	4081	607	72016	607	4074	1.04	1.00
17		608	70195	608	3937	608	68290	608	3858	1.03	1.02
18	4	901	69043	411	3625	901	64641	901	3499	1.07	1.04
19		411	66154	901	3624	411	61132	411	3487	1.08	1.04
20		645	62811	606	3377	583	59674	606	3310	1.05	1.02
21		642	78152	642	4072	642	72294	603	3920	1.08	1.04
22		603	77007	603	4058	603	72025	642	3917	1.07	1.04
23	5	604	70629	414	3732	604	67402	414	3642	1.05	1.02
24		643	66628	604	3676	643	64200	604	3576	1.04	1.03
25		414	66424	714	3571	414	63313	714	3529	1.05	1.01
26		607	71632	607	4058	607	71138	607	4036	1.01	1.01
27		608	70968	608	3977	638	60974	608	3900	1.16	1.02
28	6	638	67952	638	3532	583	60877	638	3372	1.12	1.05
29		105	65535	408	3421	608	59039	583	3343	1.11	1.02
30		582	62620	583	3357	105	58731	408	3309	1.07	1.01
31		603	78658	603	4139	603	72819	603	3985	1.08	1.04
32		642	78337	642	4083	643	72672	643	3966	1.08	1.03
33	7	643	76825	643	4078	642	70390	642	3885	1.09	1.05
34		604	72863	604	3816	604	69462	604	3709	1.05	1.03
35		411	67008	411	3748	411	62612	411	3631	1.07	1.03
36		607	71731	607	4018	607	69917	607	3977	1.03	1.01
37		608	70355	608	3986	608	68970	608	3918	1.02	1.02
38	8	901	70048	901	3712	901	66499	901	3605	1.05	1.03
39		639	66410	415	3657	639	64631	415	3599	1.03	1.02
40		414	64673	414	3642	415	62394	639	3588	1.04	1.02
41		603	78474	603	4178	603	73396	603	4037	1.07	1.03
42		643	75802	643	3996	643	72210	643	3969	1.05	1.01
43	9	604	74871	604	3945	604	71318	604	3832	1.05	1.03
44		644	68922	644	3737	644	66636	644	3643	1.03	1.03
45		899	67374	408	3552	899	62895	408	3436	1.07	1.03
46		640	71901	608	3977	608	68717	608	3913	1.05	1.02
47		607	71591	607	3963	640	68657	607	3900	1.04	1.02
48	10	639	71510	639	3884	607	68393	639	3781	1.05	1.03
49		608	69706	640	3797	639	67874	640	3704	1.03	1.03
50		411	65827	412	3729	412	63491	412	3683	1.04	1.01
51		603	79555	603	4239	603	73776	603	4078	1.08	1.04
52		644	74498	604	4014	644	70927	604	3944	1.05	1.02
53	11	643	66410	644	4001	903	64318	644	3923	1.03	1.02
54		604	66050	643	3747	604	61935	643	3714	1.07	1.01
55		904	65433	415	3558	681	61042	415	3522	1.07	1.01
56		640	75551	640	4066	640	72384	640	3946	1.04	1.03
57		901	71123	608	3932	608	68107	608	3887	1.04	1.01
58	12	607	70366	607	3887	901	67954	607	3807	1.04	1.02
59		641	69326	901	3800	607	66607	901	3704	1.04	1.03
60		608	68534	639	3576	641	63831	639	3515	1.07	1.02
61		603	79777	603	4267	604	74433	603	4109	1.07	1.04
62		644	77716	604	4104	603	73970	604	4045	1.05	1.01
63	13	604	75716	644	4010	644	68835	644	3829	1.10	1.05
64		102	70049	645	3742	645	65840	412	3659	1.06	1.02
65		899	68566	102	3697	102	65174	645	3647	1.05	1.01
66		641	77849	640	4218	641	75340	641	4107	1.03	1.03
67		640	77743	641	4185	640	72784	640	4073	1.07	1.03
68	14	103	73754	103	3901	103	67701	608	3841	1.09	1.02
69		102	70295	608	3826	608	67164	103	3749	1.05	1.02
70		607	68900	607	3806	607	64598	607	3701	1.07	1.03
71		603	79138	603	4265	604	75687	604	4135	1.05	1.03
72		604	77352	103	3930	603	73986	103	3804	1.05	1.03
73	15	645	73862	645	3820	103	68038	645	3687	1.09	1.04
74		103	72889	604	3733	104	67446	603	3648	1.08	1.02
75		904	66566	903	3629	645	65968	903	3610	1.01	1.01
76		641	81057	641	4332	641	77309	641	4236	1.05	1.02
77		901	71447	901	3863	901	68995	104	3822	1.04	1.01
78	16	105	71252	104	3850	104	68967	901	3779	1.03	1.02
79		104	68279	901	3850	608	65920	608	3777	1.04	1.02
80		902	67729	104	3783	640	62142	412	3603	1.09	1.05
81		604	79240	604	4296	604	76730	604	4214	1.03	1.02
82		105	72436	105	3920	105	68021	105	3800	1.06	1.03
83	17	899	70611	603	3705	899	64874	603	3605	1.09	1.03
84		693	66865	899	3684	603	62576	899	3539	1.07	1.04
85		603	66505	645	3549	645	62430	645	3505	1.07	1.01
86		642	82204	642	4438	642	78190	642	4307	1.05	1.03
87		105	70562	105	3776	106	66128	608	3696	1.07	1.02
88	18	581	66242	608	3690	608	64409	106	3682	1.03	1.00
89		607	64575	641	3597	681	61871	641	3557	1.04	1.01
90		608	63588	581	3525	641	61740	681	3436	1.03	1.03
91		604	80914	604	4370	604	71605	604	4280	1.13	1.02
92		903	67958	903	3678	903	63949	903	3580	1.06	1.03
93	19	904	67735	603	3611	106	63658	106	3569	1.06	1.01
94		106	66890	106	3592	638	62562	603	3554	1.07	1.01
95		638	64948	638	3584	904	61664	638	3512	1.05	1.02
96		642	84194	642	4499	642	78015	642	4321	1.08	1.04
97		901	72730	901	3942	901	69715	901	3837	1.04	1.03
98	20	682	66857	608	3616	608	62665	608	3602	1.07	1.00
99		691	65014	682	3585	682	61722	641	3480	1.05	1.03
100		581	63702	641	3472	581	60777	682	3449	1.05	1.01

* The procedures for global peak repose data points in Y-direction is the same as that in X-direction. The responses for the first 100 seconds are excluded from the largest pick response selection to get rid of transient effect.

Table 7. Natural periods for the first three modes in X and Y direction

MODE	PERIOD (SECS)	Degree of freedom
1	3.01	Y
2	1.78	X
3	1.52	X
4	1.28	Y
5	1.17	X
6	1.04	Y

Table 8. Platform description and environmental data

Overall Jacket Size at work point (EL 8.3 m)	16 (m)x36 (m)
Overall Jacket Size at mudline (EL -29.5 m)	25 (m)x36 (m)
Topside Deck dimension (EL 37.1 m)	21 (m)x40.5 (m)
Topside Deck dimension (EL 13.9 m)	21 (m)x56.2 (m)
Topside Operating Weight	5000 (MT)
Water Depth	29.3 m
Current Velocity	0.795 m/s
Wave Height	11 m
Wave Period	11.3 sec