1- Publications in Ship Structural Analysis and Design (1969-2002)

1- " Effect of Variation of Ship Section Parameters on Shear Flow Distribution, Maximum Shear Stresses and Shear Carrying Capacity Due to Longitudinal Vertical Shear Forces", European Shipbuilding, Vol. 18. (Norway-1969), Shama, M. A.,

2- "Effect of Ship Section Scantlings and Transverse Position of Longitudinal Bulkheads on Shear Stress Distribution and Shear Carrying Capacity of Main Hull Girder", Intern. Shipb. Progress, Vol. 16, No. 184, (Holland-1969), Shama, M. A.,

3- "On the Optimization of Shear Carrying Material of Large Tankers", SNAME, J.S.R, March. (USA-1971), Shama, M. A.,

4- " An Investigation into Ship Hull Girder Deflection", Bull. of the Faculty of Engineering, Alexandria University, Vol. XII., (Egypt-1972), Shama, M. A.,

5- "Effective breadth of Face Plates for Fabricated Sections", Shipp. World & Shipbuilders, August, (UK-1972), Shama, M. A.,

6- " Calculation of Sectorial Properties, Shear Centre and Warping Constant of Open Sections", Bull., Of the Faculty of Eng., Alexandria University, Vol. XIII, (Egypt-1974), Shama, M. A.

7- "A simplified Procedure for Calculating Torsion Stresses in Container Ships", J. Research and Consultation Centre, AMTA, (EGYPT-1975), Shama, M. A.

8- "Structural Capability of Bulk Carries under Shear Loading", Bull., Of the Faculty of Engineering, Alexandria University, Vol. XIII, (EGYPT-1975), Also, Shipbuilding Symposium, Rostock University, Sept. (Germany-1975), Shama, M. A.,

9- "Shear Stresses in Bulk Carriers Due to Shear Loading", J.S.R., SNAME, Sept. (USA-1975) Shama, M. A.,

10- "Analysis of Shear Stresses in Bulk Carriers", Computers and Structures, Vol.6. (USA-1976) Shama, M. A.,

11- "Stress Analysis and Design of Fabricated Asymmetrical Sections", Schiffstechnik, Sept., (Germany-1976), Shama, M. A.,

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13- "Rationalization of Longitudinal Material of Bulk Carriers, Tehno-Ocean'88, (Jpan-1988), Tokyo, International Symposium, Vol. II, A. F. Omar and M. A. Shama,

14- "Wave Forces on Space Frame Structure", AEJ, April, (Egypt-1992), Sharaki, M., Shama, M. A., and Elwani. M.,

15- "Response of Space Frame Structures Due to Wave Forces", AEJ, Oct., (Egypt-1992). Sharaki, M., Shama, M. A., and Elwani. M. H.

16- "Ultimate Strength and Load carrying Capacity of a Telescopic Crane Boom", AEJ, Vol.41., (Egyopt-2002), Shama, M. A. and Abdel-Nasser, Y.

RESPONSE OF SPACE FRAME STRUCTURES DUE TO WAVE FORCES

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ABSTRACT

In a previous paper, a method to linearize the drag force was introduced. This linearization is necessary, if the solution is to be carried out in the frequency domain. In this paper the effect of this linearization process, beside some other factors, on the deck displacement, is studied. Some of the most important factors include deck mass, current, soil properties, surface fluctuation, and relative motion between the structure and the surrounding water. The relative importance of the above mentioned factors, there effect on the structure response, seems to depend on one another.

INTRODUCTION

The finite element method is used to model the structure and to construct the mass and stiffness matrices. The structure damping matrix is taken proportional to the stiffness matrix. Also, we consider the hydrodynamic damping.

The response of any structure may be calculated using either a quasi static or dynamic analysis. Both approaches may be based on using deterministic or non deterministic methods of analysis.

The mathematical analysis of the platform result in a system of equations governing the structure motion. These equations are solved numerically in time and frequency domain to compute the structure response.

The statistics of the response for various cases is discussed. The structure response have shown deviation from Gaussian distribution in spite that surface elevation is normally distributed.

EQUATION OF MOTION

The general equation of motion for any linear elastic structure, assuming viscus damping, is given by

$$[M]\ddot{U}+[C]\dot{U}+[K]U=F \qquad (1)$$

where

$[M],[C]$ and $[K]$ the mass, damping and stiffness matrices, respectively $U, \dot{U},$ and $\ddot{U}$ the displacement, velocity, and acceleration vectors, respectively. F the force vector.

In offshore engineering it is a common practice to use a lumped mass matrix to represent the "in air" mass matrix. In this work we use an "in air" damping matrix with the form

$$[C] = \frac{2\varepsilon_1}{\omega_1}[K] \qquad (2)$$

where

ε_1 is the damping ratio of the first mode shape
ω_1 is the natural frequency of the first mode shape.

If we linearize the drag force, then the mass matrix $[M]$, the damping matrix $[C]$ and the force vector F are modified to take the form

$$[M_T] = [M]+\sum_{i=1}^{m}\rho(C_M-1)[V_i] = [M]+[M_a] \qquad (3)$$

where

$[M_a]$ is the add mass matrix
C_M is the inertia coefficient
ρ is the mass density of the water and $[V_i]$ is given by $[V_i] = 0.25\ D^2 l[N_T]$; where l_i is the length of member i and D_i is the diameter of member i.

And

$$[N_T] = 0.5\begin{bmatrix} N & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & N & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix}$$

where $[N] = [I].SS^t$; and $S = S_x I + S_y J + S_z K$ is a unit vector along the member axis; and S_{H}, S_{kb}, S_z are the direction cosines of the member axis. Finally, m is the number of members in the structure.

The damping Matrix after linearization takes the form:

$$[C_T] = [C] + \sum_{i=1}^{m} \rho C_D < |[N_{Ti}] u_i| > [A_i] \qquad (4)$$

where

C_D is the drag coefficient

u_i is a vector of order 12 representing the velocity components at the 2 nodal points of member i and $<|[N_{Ti}]u_i|>$ is the expected time average of the velocity component normal to member i,

and

$$[A_i] = D_i \, l_i \, [N_{Ti}]$$

The linearized force vector is given by

$$F = [F_M]\dot{u} + [F_D]u \qquad (5)$$

where

$$[F_M] = \sum_{i=1}^{m} \rho C_M [V_i] \qquad (6)$$

and

$$[F_D] = \sum_{i=1}^{m} .5 \rho C_D < |[N_{Ti}] u_i| > [A_i] \qquad (7)$$

$\dot{u}$ and u are the acceleration and velocity vectors of the fluid particles at the structure nodal points.

If we use the drag force in its original form, then the mass and damping matrices are reduced to the first terms given by equations (3) and (4) and the force vector is given by

$$F = \sum_{i=1}^{m} F_i$$

where

$$F_i = 0.5 \rho C_D |[N_{Ti}]\{u_i - \dot{U}_i\}| [N_{Ti}]\{u_i - \dot{U}_i\}$$
$$+ 0.25 \rho C_M [N_{Ti}] \dot{u}_i - 0.25 \rho (C_M - 1) [N_{Ti}] \ddot{U}_i$$

where u_i, $\dot{U}_i$, $\dot{u}_i$ and $\ddot{U}_i$ are vectors of order 12 representing the fluid particles velocity, the structure velocity, the fluid particles acceleration, and the structure acceleration at the 2 nodal points of member i, respectively.

FOUNDATION SYSTEM

For the purposes of dynamic response analysis, it is important to model the entire platform and its supporting foundation. The entire structure may be separated into a structure subsystem and foundation subsystem. The two subsystems are then connected together at the foundation-structure interface (bottom degrees of freedom).

The degrees of freedom at the bottom for a steel structure consist of the sum of the degrees of freedom at the nodes located at the end of all the legs. The loads are transmitted to the foundation through the interface nodal points, hence the influence of the foundation on the response of the structure can be determined using the complex stiffness matrix of the bottom degrees of freedom.

To obtain the complex stiffness matrix, a steady state dynamic analysis is carried out. The ,frequency dependent,coefficients of this matrix relate the steady state harmonic forces acting at the interface degrees of freedom, with frequency (ω) to the resulting steady state harmonic displacement.

Assuming that the stiffness and damping matrices of the structure subsystem are represented by

$$[K] = \begin{bmatrix} K_{ss} & K_{sb} \\ K_{sb}^T & K_{bb} \end{bmatrix}, \quad [C] = \begin{bmatrix} C_{ss} & C_{sb} \\ C_{sb}^T & C_{bb} \end{bmatrix}$$

where $[K_{ss}]$ and $[C_{ss}]$ are the stiffness and damping matrices associated with all the degrees of freedom, with the base degrees of freedom associated with the nodal points at the structure function interface excluded, and $[K_{bb}]$ and $[C_{bb}]$ are the stiffness and damping matrices associated with the bottom degrees of freedom, then the stiffness and damping matrices of the total structure which consists of structure and foundation systems are given by

$$[K_T] = \begin{bmatrix} K_{ss} & K_{sb} \\ K_{sb}^T & K_{bb} + K_{bb}^f(\omega) \end{bmatrix}, \quad [C_T] = \begin{bmatrix} C_{ss} & C_{sb} \\ C_{sb}^T & C_{bb} + C_{bb}^f(\omega) \end{bmatrix}$$

It may be clear that $[C_{bb}^f(\omega)]$ and $[K_{bb}^f(\omega)]$ depend mainly on the soil properties which can not be determined exactly, so it seems reasonable to use approximate methods to find out $[C_{bb}^f(\omega)]$ and $[K_{bb}^f(\omega)]$.

The methods introduced by Novak are used in this work to calculate the above mentioned matrices [9], [10], [11].

PARAMETRIC STUDY

The effect of the following parameters on the response of the tower are studied under different values of the drag and inertia coefficients and wave energy. Also, the effect of the soil characteristics on the structure response are studied. The parameters are (a) fluctuation in water surface due to the passage of waves, (b) structure motion relative to the flow, and (c) non linear form of the drag force.

Throughout the analysis, only long crested waves are considered. The direction of the wave propagation is assumed parallel to the short side of the frame. The effect of lift force (transverse force) is neglected. Finally, it is assumed that, the sea state may be described by a P-M spectrum, given by

$$S_{nn}(\omega) = (\alpha g^2/\omega^5)\, e^{-\beta (g/(\omega W))^4}$$

where

g gravity acceleration

W wind speed at 20 m above mean water level

ω wave frequency in radian per second

$\alpha = 0.0081$, $\beta = 0.74$.

To study the effect of the factors a, b, and c mentioned above, several computer programs were developed.

Each leg of the frame is supported by a single pile. Two views of the space frame used in this study are shown in Figure (1).

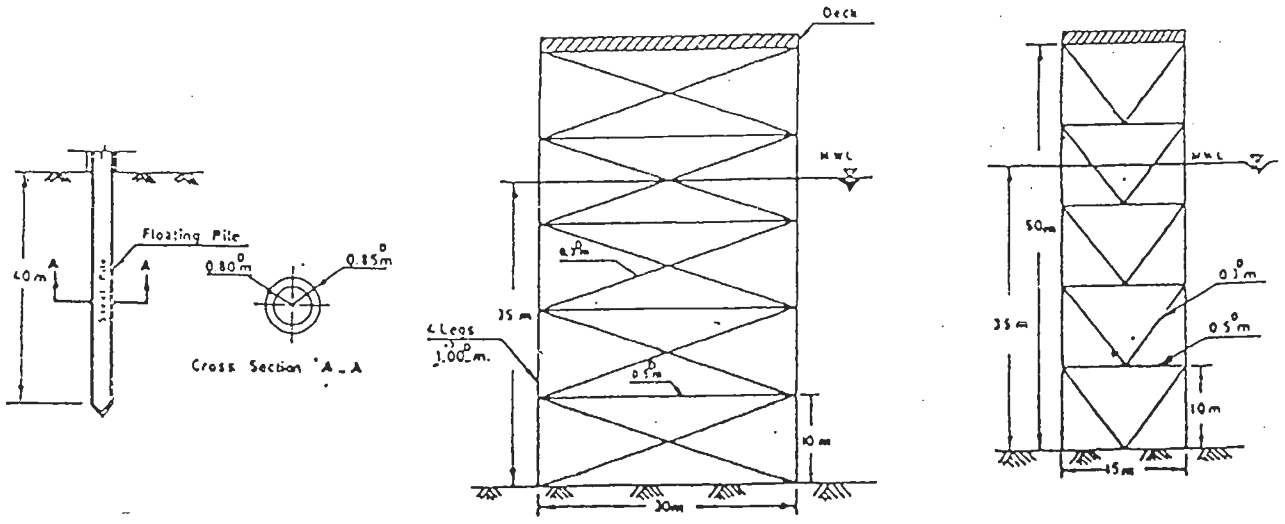

Figure 1. Short and Long Sides of the Space Frame and Details of the Pile Foundation.

EFFECT OF SURFACE FLUCTUATION AND RELATIVE MOTION ON THE RESPONSE

To study the effect of variation in surface elevation due to the passage of the wave, the frame was subjected to three wave trains with wind speeds 10, 15, and 20 m/sec. For each of these cases three values of drag coefficients (0.6, 1.0, 1.4), and three values of the inertia coefficients (1.2, 1.6, 2.0) were used. That means a total of 9 runs for each wind speed. For each wind speed the program was run for the three different cases; case a) where the effect of both the variation in surface elevation and the relation motion where taken into account, case b) where the variation in surface elevation was taken into account, but the effect of relative motion was neglected and case c) where the variation in surface elevation was neglected, and the effect of relative motion was taken into account.

The program was run using the same surface elevation and the results are shown in Figures (2-a) through (4-c). From Figures (2-a) through (2-c), it may be concluded that; in case of low wind speed, which means low wave height, neither the relative motion nor the variation in surface elevation has a significant effect on the response, for the wide range of drag and inertia coefficients used in the analysis.

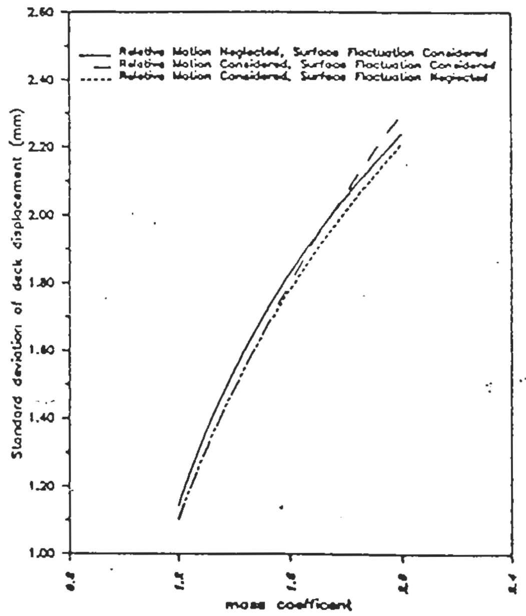

Figure 2-a. Effect of surface fluctuation and relative motion on deck displacement at wind speed 10m/sec for deck mass 12000 ton (C_D = 0.60).

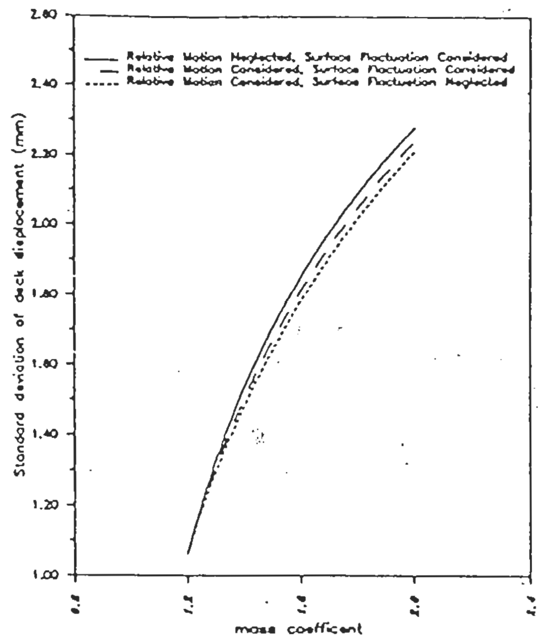

Figure 2-b. Effect of surface fluctuation and relative motion on deck displacement at wind speed 10m/sec for deck mass 12000 ton (C_D = 1.00).

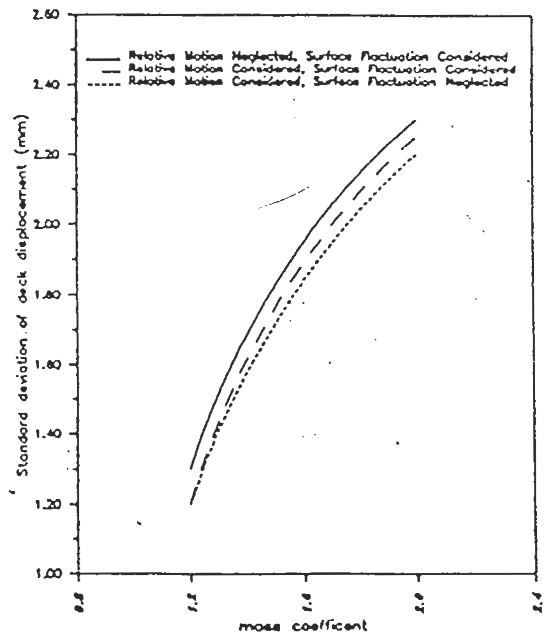

Figure 2-c. Effect of surface fluctuation and relative motion on deck displacement at wind speed 10m/sec for deck mass 12000 ton (C_D = 1.40).

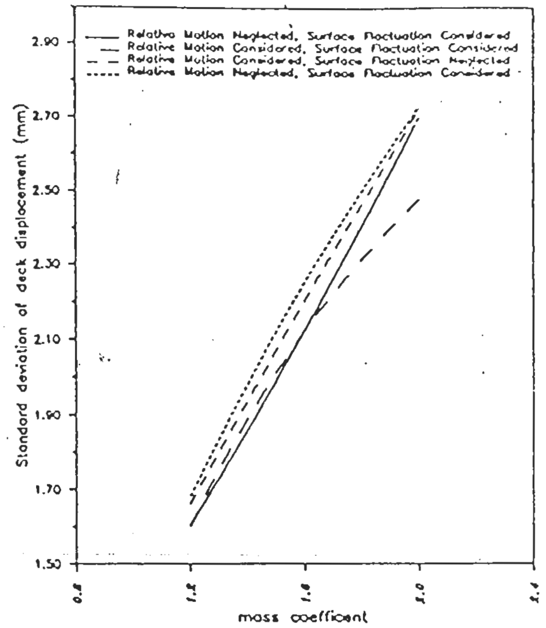

Figure 3-a. Effect of surface fluctuation and relative motion on deck displacement at wind speed 15m/sec for deck mass 12000 ton (C_D = 0.60).

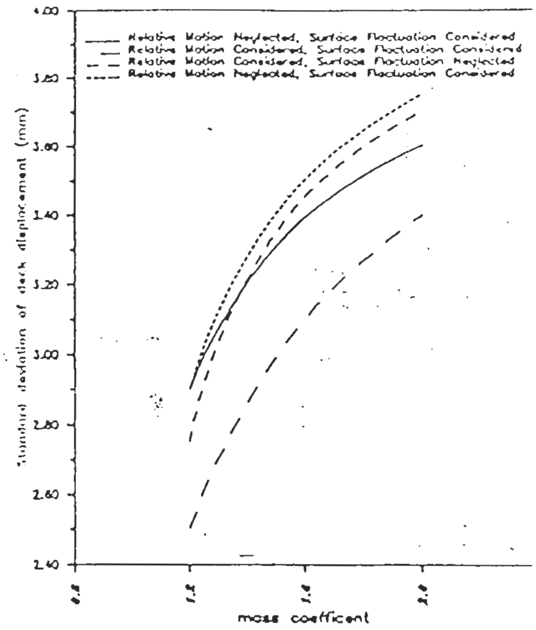

Figure 3-c. Effect of surface fluctuation and rel motion on deck displacement at wind speed 15n for deck mass 12000 ton (C_D = 1.40).

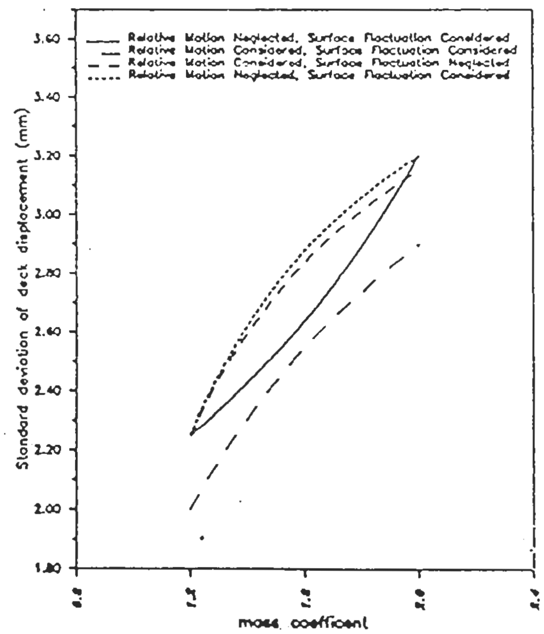

Figure 3-b. Effect of surface fluctuation and relative motion on deck displacement at wind speed 15m/sec for deck mass 12000 ton (C_D = 1.00).

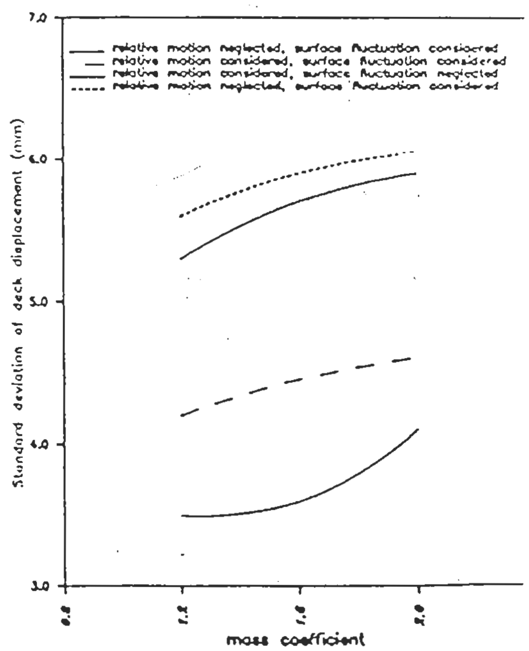

Figure 4-a. Effect of surface fluctuation and re motion on deck displacement at wind speed 20 for deck mass 12000 ton (C_D = 0.60).

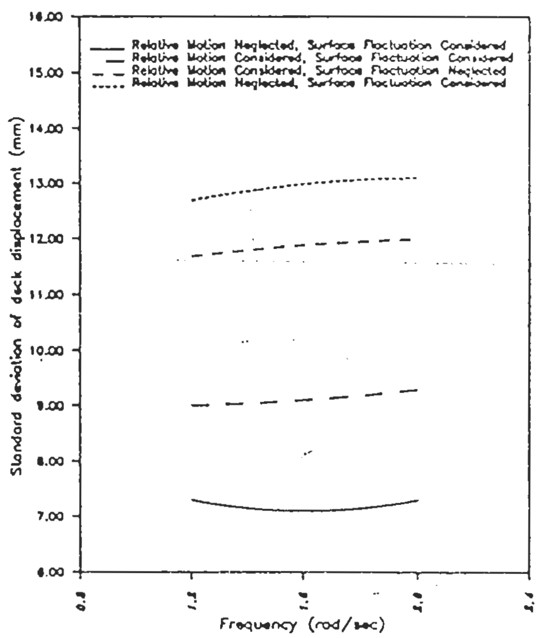

Figure 4-b. Effect of surface fluctuation and relative motion on deck displacement at wind speed 20m/sec for deck mass 12000 ton (C_D = 1.00).

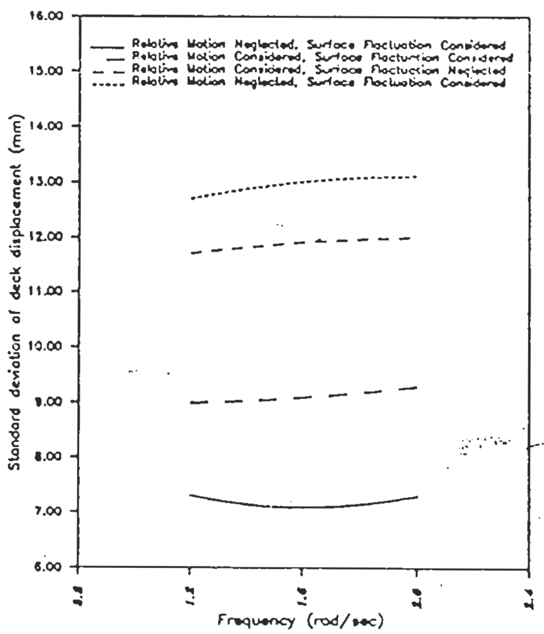

Figure 4-c. Effect of surface fluctuation and relative motion on deck displacement at wind speed 20m/sec for deck mass 12000 ton (C_D = 1.40).

From figures (3-a) through (3-c), it may be concluded that, the effect of variation in surface elevation is more important than the effect of relative motion, which has a minimal effect for the cases shown. However, for a compliant structure the effect of surface elevation and the relative motion may have significant effect on the response of the structure. This is shown when studying the effect of soil characteristics on the responses, since taking the effect of soil into account makes the structure more flexible. From Figures (4-a) through (4-c), it may be concluded that the effect of both surface fluctuation and relative motion are important. The importance of relative motion is due to increasing the total force, while the importance of surface fluctuation is due to increasing the share of the drag force.

From Figures (2-a) through (4-c), it may be concluded that, at low wind speed the structure behaves as an inertia dominant structure, while at high wind speed it behaves as drag dominant one.

Also, from Figures (2-a) through (4-c), it is clear that, using linearized drag force in predicting the response, leads to accurate results in case of inertia dominant structures, and in case of structure where both the drag and inertia forces are nearly equal. In case of drag dominant structures, determining the displacements using the linearized drag force leads to an under estimation of the displacements. This may be attributed to the large difference in forces calculated by the two methods, as shown in Table 1.

Table 1.

Wind speed 10 m/sec.

Resultant wave force on first submerged level	Linearized drag force	Non linear drag force
Mean (ton)	0.280	0.193
Standard Deviation (ton)	15.000	14.830
Skewness	$1.420 * 10^{-4}$	$8.150 * 10^{-8}$
Kurtosis	2.567	2.746
Maximum (ton)	47.220	64.720
Minimum (ton)	-42.730	-46.130

Wind speed 15 m/sec

Resultant wave force on first submerged level	Linearized drag force	Non linear drag force
Mean (ton)	2.850	2.085
Standard Deviation (ton)	42.300	37.430
Skewness	0.275	0.414
Kurtosis	2.705	2.870
Maximum (ton)	153.000	173.000
Minimum (ton)	-100.000	-108.000

Wind speed 20 m/sec.

Resultant wave force on first submerged level	Linearized drag force	Non linear drag force
Mean (ton)	13.104	10.790
Standard Deviation (ton)	488.800	80.800
Skewness	00.506	1.070
Kurtosis	2.790	5.250
Maximum (ton)	323.000	465.000
Minimum (ton)	-186.000	-198.000

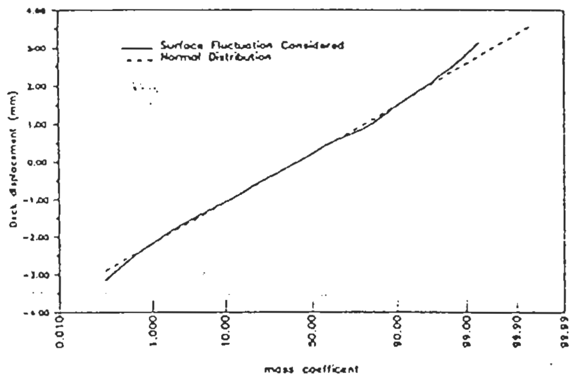

Figure 5. Cumulative distribution of deck displacement for wind speed 20 m/sec, C_D = 1.00 and C_M = 2.0.

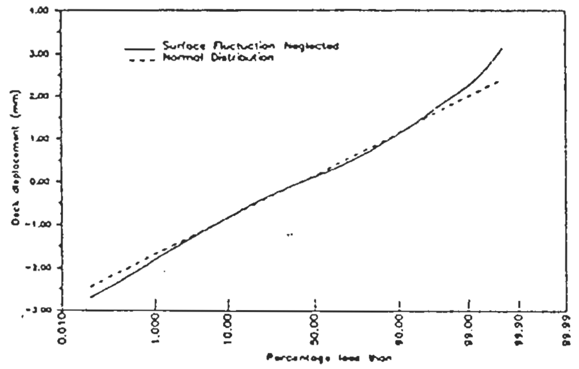

Figure 6. Cumulative distribution of deck displacement for wind speed 20 m/sec, C_D = 1.00 and C_M = 2.0.

Figures (5) and (6) show the distribution of the deck displacement for the case of wind speed 20m/sec, deck mass 12000 ton. In Figure (5) the effect of surface fluctuation is taken into account, while in Figure (6) the effect of surface fluctuation is neglected. For the case studied the surface fluctuation is shown to have no effect on the distribution of the deck displacement, although, it affects its standard deviation.

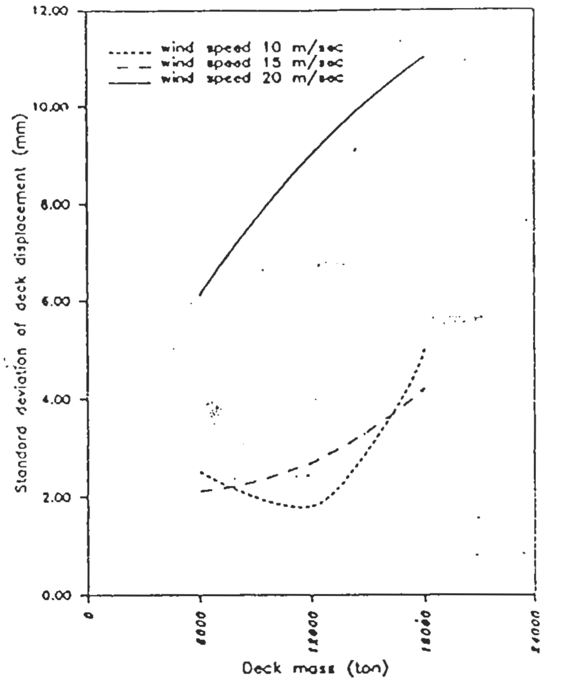

Figure 7. Effect of deck mass on deck displacemen C_D = 1, C_M = 2.

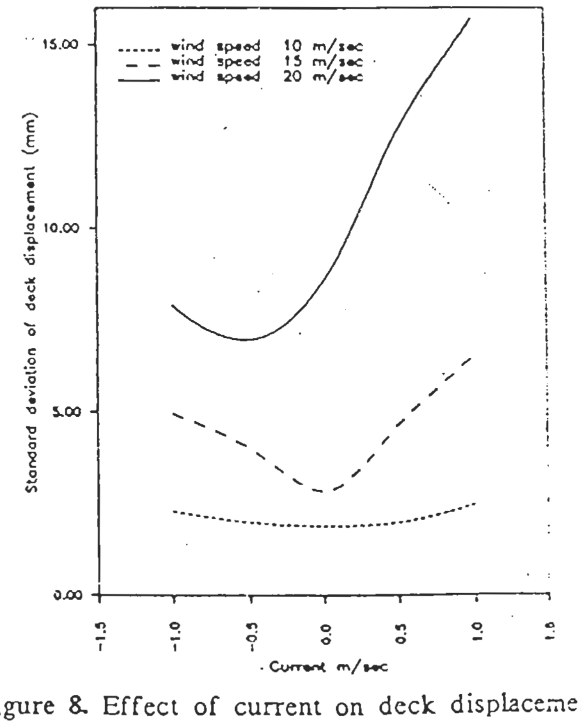

Figure 8. Effect of current on deck displacemer Deck mass = 12000 ton, C_D = 1, C_M = 2.

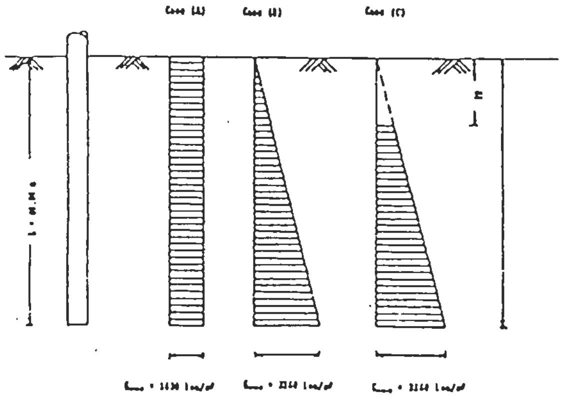

Figure 9. Variation of shear modulus (G) with depth.

EFFECT OF DECK MASS ON THE RESPONSE

To study the effect of the deck mass, the frame was subjected to three wave trains with wind speeds 10, 15, and 20 m/sec the drag and inertia coefficients were kept constants 1 and 2 respectively while the deck mass was selected to be 6000, 12000, and 18000 ton, for each wave train. The results of the analysis are shown in Figure (7). From the figure, it is clear that, there is no direct relation between the deck mass and the response. The deck mass affects the response through the variation of the natural frequency of the frame. That is, if the variation in the deck mass cause the natural frequency of the frame to be shifted so that, it comes closer to the peak frequency of the surface elevation spectrum, then resonance may occur. Although, increasing wind speed, increases the energy content of the wave and consequently the force experienced by the structure, it may cause the frequency of the applied force to be shifted away from the natural frequency of the structure, and hence reduces the structure response.

EFFECT OF CURRENT ON THE RESPONSE

To study the effect of current on the response. The frame was subjected to three wave trains corresponding to wind speeds, 10, 15 and 20 m/sec, respectively. The deck mass was 12000 ton, the drag and inertia coefficients were chosen to be 1 and 2, respectively. With each wave train, currents with vertical distribution, that varies linearly with the depth are considered, such that current at 30 m above bottom was 1 m/sec and at 20 m above bottom were 80% of current values at 30 m above bottom and currents at 10 m above bottom were 60% of current values at 30 m above bottom. The result of the analysis is shown in Figure (8).

Since the effect of current is associated with the drag force effect, and since the frame under study behave as an inertia dominant structure at low wind speed, and as a drag dominant structure at high wind speed, then it is expected that, at low wind speed the current has no effect on the response.

At high wind speed a current in the direction of wave propagation has a pronounced effect on the response. If variation in surface elevation is taken into account, and a current opposing the direction of wave propagation has less effect. If the variation in surface elevation is neglected, then it is expected that, the effect of the opposing current will be pronounced.

EFFECT OF SOIL CHARACTERISTICS ON THE RESPONSE

The stiffness of the piles, upon which the platform rests, affects, greatly the response of the platform. The stiffness of the pile, in turn, is a function of the soil characteristics. The soil characteristics that affect the pile stiffness include; Poisson's ratio, soil material damping, soil shear modulus, and the method of variation of shear modulus with depth. Two types of variation in shear modulus are considered. In the first case the shear modulus is assumed to be uniformly distributed. In the second case the shear modulus is assumed to vary linearly with depth, as shown in Figure (9).

Another important factor that affects the stiffness of the pile is the separation between pile and the soil near mud line. The soil pile separation at the sea floor is caused by the cyclic movement of the structure, and scouring caused by currents. The soil up to a depth equal to twice the outer diameter of the pile is assumed to cause no resistance to the pile movement. The shear modulus is assumed to vary linearly with depth as shown in figure (9).

Hereafter, the effect of two factors are studied; variation in shear modulus with depth, cases A and B (Figure 9) and separation between pile and soil case C, Figure (9). Table (2) summaries the properties of soil used in this study.

Table 2. Properties of Soil Used in the Analysis

Average shear modulus (ton/m^2)	1630
Maximum shear modulus (ton /m^2)	3260
Mass (t/m^3	1.82
Poisson's ratio	0.50
Material damping	0.10

At wind speed 10 m/sec the highest wave component included in the analysis has frequency 1.68 rad/sec, the pile damping in his case is nearly zero. Table (3) gives the value of pile stiffness used in the analysis.

Table 3. Stiffness of the pile foundation system used in the analysis

Cases	Horizontal	Rocking	Coupling
A: Uniform shear modulus	2.29 E4	1.29 E5	-3.76 E4
B: Linearly varying shear modulus	5.94 E3	9.33 E4	-1.82 E4
C: linearly varying shear modulus with gapping	3.69 E3	8.45 E4	-1.44 E4

Table (4) gives the natural frequencies of the frame with deck masses 6000, 12000, and 18000 ton for the three cases A, B, and C of the soil described above.

Table 4. Natural Frequencies of the plane frame with soil represented by Cases A, B, and C (rad/sec).

Soil Case A

Deck mass (ton)	Natural frequency		
	First	Second	Third
6000	1.33	6.08	12.81
12000	1.00	5.87	12.71
16000	0.83	5.80	10.86

Soil case B

Deck mass (ton)	Natural frequency		
	First	Second	Third
6000	0.91	5.10	12.06
12000	0.70	4.85	11.96
16000	0.59	4.75	10.86

Soil case C

Deck mass (ton)	Natural frequency		
	First	Second	Third
6000	0.76	4.91	11.94
12000	0.59	4.64	11.84
16000	0.49	4.54	10.86

Table (5) shows the statistics of the deck displacement for wind speeds 10, 15, and 20 m/sec respectively, foundation systems with the properties introduced in A, B, and C above are chosen for the analysis. The deck mass is taken to be 12000 ton with the drag and inertia coefficients 1, and 2 respectively.

Table 5. Effect of soil properties on the statistics of deck displacement, for wind speed 10 m/sec, 15 m/sec, and 20 m/sec. Deck mass is 12000 ton C_o and C_M are 1 and 2 respectively.

Wind speed 10 m/sec.

Deck mass displacement	Fixed base	Soil case A	Soil case B	Soil case C
mean (m)	5.82 E-6	2.01 E-6	1.56 E-5	3.56 E-6
Standard deviation	1.86 E-3	1.87 E-2	1.21 E-2	2.43 E-2
Skewness	3.56 E-3	-6.30 E-4	3.20 E-3	1.89 E-2
Kurtosis	2.54	1.93	2.32	1.92
Maximum (m)	5.32 E-3	4.07 E-2	3.22 E-2	6.02 E-2
Minimum (m)	-5.50 E-3	-4.06 E-2	-3.09 E-2	-7.71 E-2
Zero period Crossing (sec)	5.45	6.32	6.53	10.40

Wind speed 15 m/sec.

Deck mass displacement	Fixed base	Soil case A	Soil case B	Soil case C
mean (m)	5.82 E-6	9.09 E-6	3.90 E-5	2.41 E-4
Standard deviation	2.89 E-3	2.00 E-2	0.174	0.175
Skewness	1.18 E-2	-7.67 E-4	1.10 E-3	6.35 E-4
Kurtosis	3.00	2.10	1.65	2.13
Maximum (m)	1.19 E-3	4.96 E-2	0.38	0.39
Minimum (m)	-1.03 E-3	-6.06 E-2	-0.38	-0.39
Zero period Crossing (sec)	6.47	6.52	9.09	10.70

Wind speed 20 m/sec.

Deck mass displacement	Fixed base	Soil case A	Soil case B	Soil case C
mean (m)	3.25 E-4	6.14 E-4	1.43 E-3	1.97 E-3
Standard deviation	8.11 E-4	2.27 E-2	0.11	0.295
Skewness	1.39 E-2	7.18 E-2	-1.63 E-2	-1.13 E-3
Kurtosis	4.48	3.49	2.61	2.23
Maximum (m)	4.69 E-2	8.38 E-2	0.32	0.71
Minimum (m)	-5.03 E-2	-0.10	-0.33	0.70 E-2
Zero period Crossing (sec)	5.50	7.47	9.50	10.90

In all the cases tested, it is found that reducing the stiffness of the pile results in increasing the deck displacement of the platform. Except for the cases A and B with wind speed 10 m/sec.

This may be explained as follow. The first natural frequency of the frame for cases A and B are 1 and 0.7 rad/sec respectively. The peak frequency of the surface elevation spectrum at wind speed 10 m/sec is 0.86 rad/sec, nearly midway between the two natural frequencies. However, if the frequency of the applied force is less than the natural frequency of the structure, then its dynamic effect is greater than that of a force having frequency greater than the natural frequency of the structure. Assuming that the natural frequency of the structure is nearly midway between the frequencies of the force. [12]

In analyzing the frame with the base fixed it was found that the effect of relative motion on the response has nearly no effect. However, with a flexible structure, this may not be the case. The frame was subjected to a wave train with wind speed 20 m/sec the foundation is supposed to be represented by case C introduced above. The deck displacement where calculated twice. Once with the effect of relative motion neglected and once with the effect taken into account. The values of the deck displacement standard deviation where 0.33 and 0.28 m, respectively. This show the importance of considering the effect of the relative motion in case of flexible structures.

CONCLUSIONS

From the results obtained, we conclude the following:

1. For drag dominant structures there are needs to consider the effect of currents, free surface fluctuation and to use the non-linear form of the drag term to compute accurate structure response.
2. The non-linear drag term of Morison equation causes a deviation of the response distributions from Gaussian, as indicated by Kurtosis value, for these distributions greater than 3. Thus the non-linear drag forces increase the probability of extreme values of structure response.
3. In case of linearizing the drag term of Morison equation, the distribution of displacement is Gaussian, if we ignored surface fluctuations. For the space frame presented the linearized Morison equation were accurate in computing the deck displacement at wind speeds 10 and 15 m/sec and underestimated the deck displacement for wind speed 20 m/sec.
4. For the specific space frame considered, we noticed that free surface fluctuation has no effect on the Skewness of the distribution of the deck displacement. The standard deviation of deck displacement was larger for the case in which we considered surface fluctuations, see Figures (5) and (6).
5. Deck mass has pronounced effect on the deck displacement. It varies the natural frequencies of the space frame so that, the first mode frequency was shifted closer to the peak frequency of the force spectrum.
6. Soil characteristics have important effects on the response of the structure, not only through changing its stiffness but also through changing natural frequencies and bringing them closer to the peak frequency of wave force.
7. The effect of current on the structure displacement should be considered especially when current direction coincide with the direction of wave propagation and surface fluctuation is taken in consideration.
8. In computing the response of flexible structures the motions of the structure relative to the flow field should be considered.
9. Frequency domain solutions were much faster than time domain solutions. For example, we have noticed that for the space frame considered, the time ratio between the two methods was about 1:30. The frequency domain solutions require linearization of the drag force term of Morison equation. In addition it is difficult to incorporate the effect of currents in response computation.
10. In the frequency range between 0 and 2 rad/sec the stiffness of foundation system may be taken constant.

From the results obtained the following is recommended:

1. Offshore structures may be classified either drag dominant or inertia dominant structures. The

classification must be carried out for all the ranges of wind speeds, wave energy, and for all the possible combinations of the drag and inertia coefficients. The ratio between the variance of the force caused by drag and inertia forces, taken separately may be considered as a measure for the classification.
2. For inertia dominant structures the full analysis may be carried out in the F.D. Also, this is true for drag dominant structures, when the variation in surface elevation due to passage of waves and relative motion of the structure with respect to the flow can be neglected.
3. The method introduced to linearize the drag force may be used to calculate the wave forces. However its validity in estimating the structure response is limited to the cases of inertia dominant structure.
4. The current effect is important specially for drag dominant structures.
5. The response of the structure must be evaluated for the full range of the deck masses.
6. In case of drag dominant structure the analysis must be carried out in the Time Domain.
7. If the structure is narrow band, then the calculation of the response is sensitive to the number of wave components used in the analysis and further studies is needed to determine the proper number of wave components to be used.

For further research work we recommend considering the effect of wave directionality on the structure response computation. Further the procedure presented here need to be verified experimentally in a random wave tank or versus measurements obtained from random seas. It appears from this study that deck mass and foundation system play an important role in the dynamic performance of offshore structure. Time domain solutions need to be developed such that foundation system stiffness and damping properties can be incorporated in the solution procedure or a combined time and frequency domain should be developed to take the advantages of both The computer programs presented in this study can be used to study the effect of the failure of a member or a group of members on the response of the structure.

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