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EFFECTS OF SPATIAL VARIATION OF EARTHQUAKE MOTIONS ON PLATFORM RESPONSE

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SUMMARY

Influence of the spatial variability of earthquake motion on response of large offshore gravity base platforms is studied in this paper. The seismic environment is described as a random field varying in both space and time. Evaluation of stochastic kinematic interaction is described in detail and responses of two platforms are evaluated. Other aspects of the spatial variation of earthquakes that might be of interest for offshore oil fields are also discussed.

INTRODUCTION

Earthquake records obtained from dense arrays reveal some degree of variability over relatively short distances. The large foundation dimensions of the offshore gravity platforms in the North Sea mean that the spatial variability of the seismic environment may result in significant filtering of high frequencies of the effective base motion. Furthermore, the lateral variability of horizontal earthquake motion induces a torsional base movement that would otherwise not exist.

Earlier studies related to this topic (e.g. Refs 1, 2, 3, 4) have mainly focused on the changes in the effective base motion caused by the spatial variability of motion. In this paper, the degree of importance of the filtering of high frequencies on the dynamic response of a typical large North Sea gravity base platform is studied. The magnitudes of expected induced torsional base motions are also estimated. In the final section of the paper, the consequences of the spatial variability of earthquake motions on seismic reliability of a group of platforms at an offshore site are briefly discussed.

SEISMIC ENVIRONMENT

The lateral spatial variation of ground motion is a result of non-vertical propagation of body-wave energy, surface-wave propagation, waves arriving from different points on an extended source, and amplitude changes and time delays due to inhomogeneities along the propagation path. The first two factors, i.e. non-vertical incidence of body-wave energy and surface-wave energy, are usually referred to as "deterministic effects", and the other factors are referred to as "random effects" (Ref.3). Since the deterministic effects are not believed to be significant in the North Sea environment (Ref. 5), only the random effects are considered in this paper. It should be noted that the seismic activity level of Norway and its continental shelf is comparable to that of the eastern part of the USA, with historical earthquakes with magnitudes up to $M_S = 6.2$ having occurred during the last 200 years (Ref. 6).

To describe the variability of the design earthquake in both space and time, the free field motion was modelled as a random field of displacements, U, with a cross spectral density function of the following form:

$$S_{U_i U_j}(\underline{x},\omega) = \gamma(\underline{x},\omega) \cdot S_{U_0}(\omega) \tag{1}$$

where $S_{U_i U_j}$ is the cross-spectral density function between motions at points i and j, $\underline{x}$ is the vector of separation distance, ω is the frequency, $S_{U_0}(\omega)$ is the power spectral density function of earthquake motion at a point, and $\gamma(\underline{x},\omega)$ is the coherency function.

Two types of coherency functions were used in the study: Type 1, the model recommended in Ref.7 based on the data obtained from the SMART-1 array in Taiwan; and Type 2, the exponential model of the type recommended in Ref. 1. These two models of the coherency function are compared in Fig. 1 for two separation distances. The homogeneity of the soil deposit at the SMART-1 array area implies that random variability of soil parameters does not account for the rapid decrease of coherency of earthquake motions in space.

EVALUATION OF KINEMATIC INTERACTION

The foundation substructure is usually much more rigid than the underlying soil. When the earthquake-induced movements vary across the foundation contact area, the stiff structural foundation cannot conform to the soil deformations. This effect, which is known as "kinematic interaction", will cause some filtering of the high-frequency translational motions, and may also give rise to rotational foundation motions.

The exact solution to the kinematic interaction problem is very complex, and analytical solutions are available only for a few, highly-idealized situations. The approximate solution suggested by Iguchi (Ref. 8) for a rigid foundation was therefore adopted in this study. This solution, which was originally proposed for deterministic analyses, can be summarized by the following equation:

$$\underline{u} \cong [H]^{-1} \int_S [A]^T \underline{u}^f \, dS - [K]^{-1} \int_S [A]^T \underline{T}^f \, dS \tag{2}$$

where $\underline{u}$ is the vector of foundation movements, S is the foundation contact area, $\underline{u}^f$ and $\underline{T}^f$ are vectors of free field displacements and stresses along the contact area respectively, [K] is the dynamic stiffness matrix of the foundation, [A] is the rigid body transformation matrix defining the motion at any point on the soil foundation interface, and [H] is $\int_S [A]^T [A] \, dS$. Further details of this approximate solution may be found in Refs 2 and 8. For a cylindrical, embedded foundation (Fig. 2), each component of the foundation motion from Eq. (2) has the following form:

$$u = \sum_{i=1}^{2} \int_{S_i} [g_i \, u_f(\underline{x}_i) + f_i(\underline{x}_i) \, \tau_f(\underline{x}_i)] \, dS_i \tag{3}$$

where g_i and f_i are functions that depend on the component of motion being evaluated, $(\underline{x}_i)$ denotes a function of position, and u_f and τ_f are the free field displacements and stresses, respectively. Evaluating the Fourier transform of u, one obtains:

$$U = \sum_{i=1}^{2} \int_{S_i} [g_i \, U_f(\underline{x}_i) + f_i(\underline{x}_i) \, T_f(\underline{x}_i)] \, dS_i \qquad (4)$$

where U_f denotes the Fourier transform of u_f and T_f denotes the Fourier transform of τ_f. The power spectral density of u can be obtained by:

$$S_{UU} = C \cdot E[U\bar{U}] \qquad (5)$$

where $E[\cdot]$ signifies the expected value, $\bar{U}$ is the complex conjugate of U, and C is a constant. Substituting Eq. (4) into (5), and changing the order of integration and expectation, one obtains:

$$S_{UU} = \sum_{i=1}^{2} \sum_{j=1}^{2} \int_{S_i} \int_{S_j}$$

$$[g_i \, \bar{g}_j \, S_{U_f U_f} + g_i \, \bar{f}_j \, S_{U_f T_f} + f_i \, \bar{g}_j \, S_{T_f U_f} + f_i \, \bar{f}_j \, S_{T_f T_f}] \, dS_i \, dS_j \qquad (6)$$

where $S_{U_f U_f}$ is identical to $S_{U_i U_j}$ in Eq. (1). The other cross spectra in Eq. (6) were obtained by assuming the seismic field to consist of vertically propagating waves in either a homogeneous soil profile (idealization of Sleipner site), or in a soil profile whose shear modulus increases linearly with depth from a value of zero at the surface (idealization of Troll site). The integration in Eq. (6) was carried out numerically using the Gauss formulas.

EVALUATION OF GRAVITY PLATFORM RESPONSE

Having obtained the power spectrum of the effective base motion by Eq. (6), the power spectrum of the platform response of interest may be evaluated using the 3-step solution as described in Ref. 9 and depicted in Fig. 3. Both the evaluation of kinematic interaction using Iguchi's approximation and the 3-step solution require that the dynamic, frequency-dependent foundation stiffness matrix is known. This stiffness matrix was evaluated numerically using the finite element program BIAX (Ref. 10) for the platforms studied. The structural response was evaluated using the lumped mass model described in Ref. 11. The maximum earthquake-induced platform response values were estimated from the power spectra using the procedure described in Ref. 12.

EXAMPLE PROBLEMS

Two concrete platform concepts developed by Norwegian Contractors a.s were used in the study. The first platform is suggested for the Sleipner site and the second platform is suggested for the Troll site. Both sites are in the North Sea. Table 1 compares some of the key data for these two platforms.

Table 1 Key data for platforms studied

Platform site	Foundation area (m²)	Embedment (m)	Water depth (m)	Soil description
Sleipner	14600	~0	82.5	Dense sand and stiff clay
Troll	18680	33	305.0	Soft clay

Figures 4 and 5 show the absolute values of transfer functions for horizontal and torsional foundation motions respectively. The platform response values evaluated were deck acceleration, base shear and base overturning moment. Compared to the response values for perfectly coherent free field motion, the maximum response values were reduced by about 4-5% for coherency function of Type 1, and by 6-9% for coherency function of Type 2.

COMMENTS AND DISCUSSION

The amount of reduction in maximum response values, caused by the spatial variation of motion across the foundation, was rather small for the platforms studied. The reason is that the platform response values considered were mainly caused by the low frequencies of the earthquake motion (i.e. response of the lower modes). This can be seen, for example, from the transfer functions for deck acceleration in Fig. 6. All available coherency models for earthquake motions, including the ones used in this study, predict very little loss of coherency at low frequencies (i.e. $\gamma(r,\omega) \approx 1$ for small ω's). The different results for the two coherency functions are predictably related to the magnitude of $\gamma(r,\omega)$. The effects of accidental torsional base motion have not been separately evaluated.

An interesting aspect of the spatial variability of earthquake motions over short distances is its implications for the seismic safety of a group of platforms at an offshore field. From a seismic safety standpoint, especially for long-period structures, this aspect is potentially more significant than the issue of response reduction due to local partial coherence studied in this paper. The distances among "n" functionally connected structures in an oil field may cover distances of, say, 1 to 5 kilometers. Conventional attenuation laws predit negligible differences in ground motion over such distances; yet available strong motion array data indicate that there is substantial variability in peak accelerations, velocities, and (response) spectral amplitudes over such distances. This has clear implications for "system (un)reliabilities", for example, the probability that the design acceleration or design response spectral amplitude will be exceeded for all "n" structures in the field (given the occurrence of a design level ground motion). Looking further into this aspect of ground motion variability is strongly recommended for future studies.

ACKNOWLEDGEMENT

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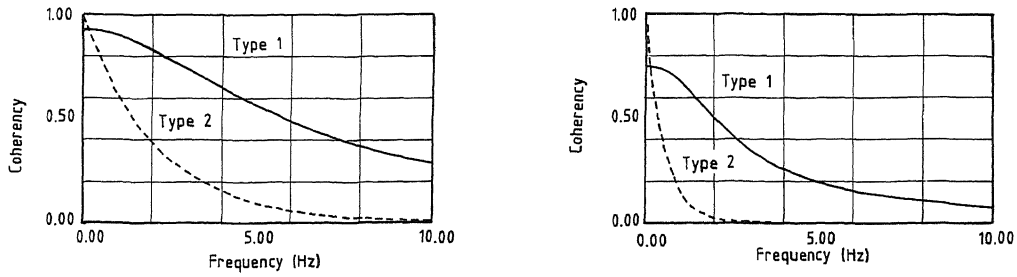
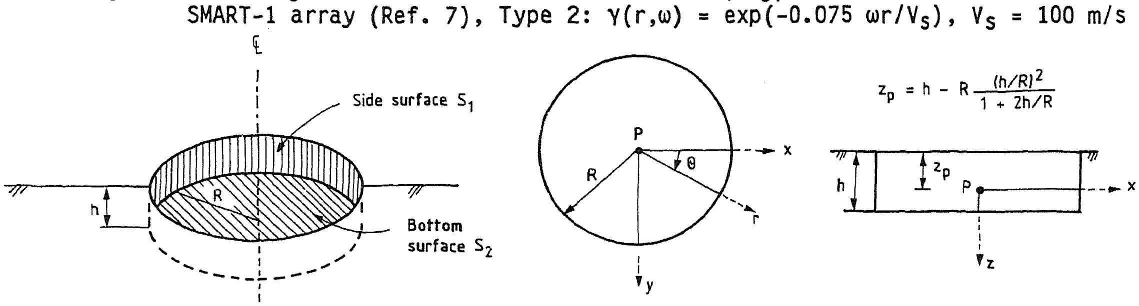


a) Separation distance = 100m b) Separation distance = 400m

Fig. 1 Coherency functions for free field motion, Type 1: based on data from SMART-1 array (Ref. 7), Type 2: $\gamma(r,\omega) = \exp(-0.075\ \omega r/V_S)$, V_S = 100 m/s



a) Cylindrical, embedded foundation b) Top view c) Side view

Fig. 2 Cylindrical, embedded foundation considered in the analyses

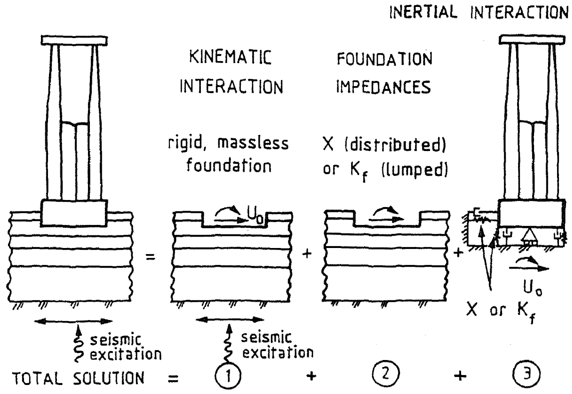
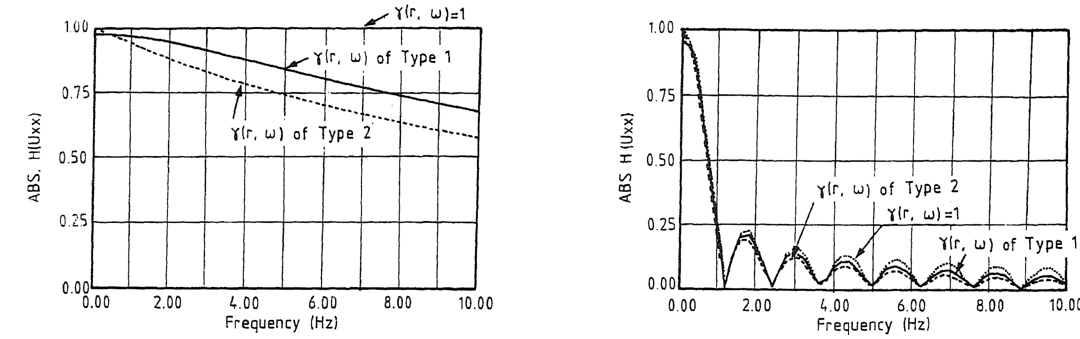


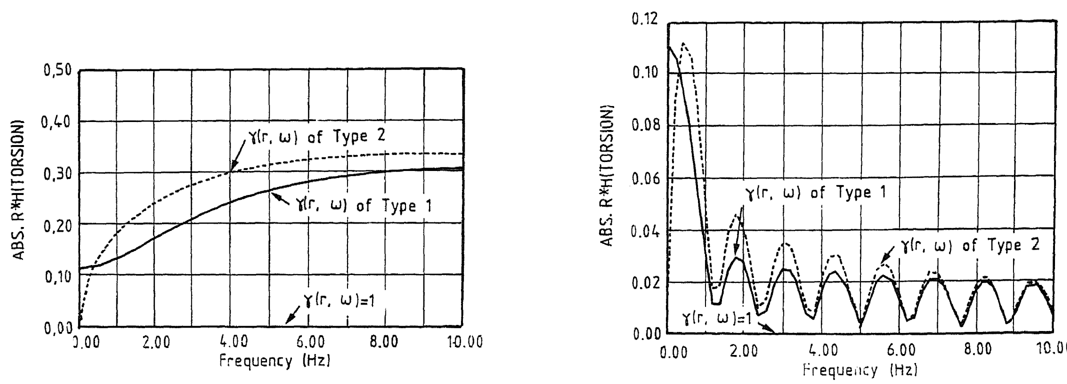
Fig. 3 The 3-step solution for soil-structure interaction



a) Sleipner platform

b) Troll platform

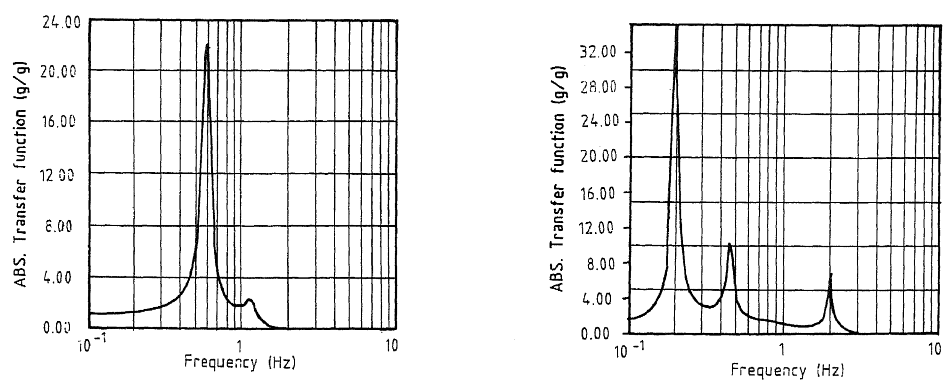
Fig. 4 Absolute values of transfer functions for horizontal motion of the foundation centroid (point p in Fig. 2) considering different coherency models



a) Sleipner platform

b) Troll platform

Fig. 5 Absolute values of transfer functions for torsional foundation motion considering different coherency models



a) Sleipner platform

b) Troll platform

Fig. 6 Absolute values of transfer functions for deck acceleration