THE EFFECT OF EARTHQUAKES ON THE FOUNDATION STABILITY OF GRAVITY OIL PLATFORMS

by

R.Dungar* and P.J.L. Eldred**

SUMMARY

In many cases gravity platforms are analysed only for static loads, but the existence of a massive, relatively rigid structure resting on a weak foundation material poses a problem which is potentially of a dynamic nature. This paper presents results for the response of such a structure, analysed by the finite element method, and including boundary dashpots to represent the energy radiated from the structure. The problem of reflecting surfaces caused by foundation layering is investigated; and the response of the structure to dynamic earthquake loading is compared with that predicted for dynamic wave loading.

INTRODUCTION

The topic of gravity platform stability is currently of great interest in many parts of the world, especially in the North Sea. This particular area has necessitated the drilling of wells in ever increasing depths of water, for which the large, gravity type platform has become popular. It is believed that this trend to increased platform size has greatly increased the response of the platform-foundation system, together with the surrounding water, to both earthquake and wave loading. The case of wave loading has been investigated for several platforms by Dungar and Eldred (1976) and Dungar, Eldred and Severn (1976), in which it was concluded that in these cases, the platforms could be analysed in a static manner for North Sea wave loading. However, if a very weak foundation is chosen with horizontal reflecting boundaries between soil layers, there is the possibility of the overall D.M.F. (Dynamic Magnification Factor equals the maximum dynamic displacement divided by the maximum static displacement, both using the same load amplitudes) being as large as five. In the following sections, some of the platforms analysed for wave loading are re-investigated for the case of earthquake loading.

EARTHQUAKE RECORDS

The area of the North Sea is normally considered to be seismically inactive. However, several earthquakes have been noted during the past 200 years; and the increased activity involved in the extraction of oil and gas from below the sea bed, is liable to alter its geological stability. Consequently there is a real possibility of an earthquake occurring somewhere within the operating area during the lifespan of those gravity platforms currently under construction and in service.

When considering earthquake loading, the first problem is that of choosing an appropriate design earthquake or appropriate earthquake parameters for a region where no records exist. Mallard et al (1975) have investigated seismic design criteria for England and Wales; from which a reasonable maximum ground acceleration to be expected, and to be designed for, is 0.2g (Richter Magnitude 5.5 at a focal distance of 10 km). Generalised acceleration and displacement spectra have been extracted from

* Lecturer in Civil Engineering, University of Bristol, England
** Engineer, Geocomp (U.K.) Limited, England

Newmark and Rosenblueth (1971), and are replotted for two damping ratios in Figures 1 and 2. In Figure 1, the spectral acceleration has been plotted to a base of unit maximum ground acceleration. This is equal to the earthquake D.M.F. (the maximum structural dynamic response for a given, varying ground acceleration record, divided by the equivalent maximum static response). Figure 2 gives a similar plot for the spectral displacement, which illustrates that the maximum structural displacement response occurs when the system has a 'resonant period' of approximately 4 secs. These figures will be used to estimate the platform response for earthquake loading, after the calculation of the 'resonant period' and damping ratio of the platform-foundation system.

GROUND RESONANT PERIOD

The term 'resonant period' for a structure such as a gravity platform resting on a semi-infinite foundation must be used with care. This system is not a truly resonant system as it is not contained within finite reflecting boundaries. Most of the energy applied to the structure is radiated away in the form of compression, shear and Rayleigh waves. However, when the response due to a sinusoidally varying load is plotted against the period of this load, there exists a period for which the response is a maximum. This will be referred to as the ground resonant period, Tg. Additionally, this plot for the first mode of vibration, resembles that of a single degree of freedom system. Consequently, an approximation to the single degree of freedom system parameters can be made by equating Tg to the resonant period, and the maximum structural D.M.F. (the maximum dynamic response to sinusoidal loading divided by the maximum static response) to 1/2c, where c is the damping ratio.

Several methods have been used in the past to obtain Tg, as described by Dungar and Eldred (1976). However, they suffer from either an inability to represent a general foundation condition, or from an artificial termination of the foundation at a finite, reflecting boundary, which in turn creates an incorrect, true resonant condition. The method used in the above paper will again be used here, and will now be briefly described for completeness.

METHOD OF ANALYSIS

In this study, the approach used is based upon the finite element method, but includes a representation of the energy radiated away from the structure and associated foundation. Finite elements are also used for the fluid region.

The axisymmetric elements used to represent both the structure and the foundation are of triangular cross-section, with the stresses and strains assumed to be harmonic functions of the polar co-ordinate, θ; as described for static analysis by Wilson (1965). Application to dynamic analysis, and in particular to earthquake loading, has been discussed by Dungar (1972). This axisymmetric geometry reduces the degree of numerical complexity and size as compared with an equivalent, full three-dimensional analysis. As the foundation is in many instances nearly axisymmetric, this idealisation tends only to affect the stresses and displacements within the structure itself, and gives a good estimate for the large dynamic stresses within the foundation.

The foundation properties, as well as some properties within the structure, are elastic over a limited range of stress. However, only linear elastic properties are used for this study, as a full non-linear analysis is beyond the scope of the present paper. This method, however, will serve to indicate the general trends, as will be described later. In order to avoid the problem of reflecting foundation boundaries, the foundation mesh is given finite boundaries, to which damping terms are added to represent the energy radiation in the prototype situation. This method is based on that of Lysmer and Kuhlemeyer (1969), and is discussed by Dungar and Eldred (1976).

Axisymmetric finite elements are also used to represent the fluid region. Here the effects of the compressibility of the fluid, surface waves and fiscosity are neglected, to enable the calculation to be made for the fluid add-mass. This problem reduces to the solution of the Laplace equation, general details of which are given by Zienkiewicz (1971).

Earthquake forces are described for the structure by assuming that the foundation would act as a rigid body, with free field acceleration a(t), if the structure was not present. In order to obtain the variation from this free-field condition, a generalised force vector, equal to the mass of the structure times a(t) is considered to act, see Clough and Penzien (1975). The assumption of rigid body, free-field motion is reasonable as long as the size of the structure is small compared with the wave-length of the ground motion, which is true in our case for frequencies of up to approximately 1 Hz. For higher frequencies, a more complicated assumption should be made, which would not be of the required non-axisymmetric, harmonic form.

DISCUSSION OF RESULTS

Two different foundation conditions were studied for a range of sinusoidal forcing frequencies from 0.15 to 5 Hz (0.2 to 7 second period). The first had elastic modulus 3×10^4 kN/m^2 and density 2×10^3 kg/m^3; and the second was identical except for a hard layer of elastic modulus 3×10^5 kN/m^2 from 30m to 50m depth. An idealised platform of height 120m, base diameter 120m, elastic modulus 2×10^7 kN/m^2 and mass 8×10^8 kg was used for both studies. Results of structural D.M.F., plotted against forcing frequency, are shown in Figures 3 and 4 respectively. In each case, two curves are shown; (i) refers to the horizontal motion of the sea bed at the edge of the structure, and (ii) refers to the vertical motion at the same point.

For the homogeneous case (Figure 3) the equivalent resonant period, obtained by averaging (i) and (ii), is approximately 5.5 secs, and the damping ratio 18%. Similarly for the layered case, the values are 4.2 secs and 16%. From Figure 1, the earthquake D.M.F.s are approximately 0.07 and 0.12 respectively, and the maximum displacements are approximately equal to the maximum ground displacements. When the estimated maximum ground acceleration of 0.2g is used, the dynamic amplitude of maximum principal stresses in the foundation are found to be 30 kN/m^2 and 45 kN/m^2 respectively, as compared with equivalent wave loading results of 240 kN/m^2 and 200 kN/m^2.

In contrast to this, when the Christchurch Bay platform of Dungar, Eldred and Severn (1976) is reconsidered for earthquake loading, two resonant periods of 0.73 secs and 0.53 secs, damping ratios 6% and 2% respectively, are obtained. In this case, the earthquake D.M.F.s are 1.6 and 3.3 respectively, but the maximum displacements, in both cases, are only 0.35 times the maximum ground displacements. Consequently, the maximum principal stress amplitudes are 32 kN/m^2 and 65 kN/m^2 respectively, as compared with the wave loading result of 13 kN/m^2 for both cases.

These two examples give two extremes. The first (the idealised platform) gives a condition where a static analysis would severely overestimate the dynamic response, whereas the second (Christchurch Bay platform) gives a condition where the static analysis underestimates the dynamic response. Similarly, the first platform would be safe for earthquake loading if the structure had been designed only for dynamic wave loading, whereas the second would not be safe. As far as designing for earthquake loading is concerned, in both cases dynamic, rather than static, analyses should be carried out, in order not to over-design or under-design.

CONCLUSION

When analysing gravity platform structures for conditions of dynamic loading, the foundation should not be terminated at an artificial boundary without allowing for the transmission of energy across this boundary. Different types of dynamic loads, for example earthquakes and waves, have different critical periods; so a full analysis should allow for both of these loading conditions. In most cases, a simple static analysis alone is insufficient for an economical and safe design.

REFERENCES

CLOUGH R.W. and PENZIEN J. (1975) Dynamics of Structures. McGraw Hill.

DUNGAR R. (1972) The Three-Dimensional Analysis of Structural Foundations During Strong Motion Earthquakes. 4th European Symposium on Earthquake Engineering, London. pp 25-39.

DUNGAR R. and ELDRED P.J.L. (1976) The Dynamic Response of Gravity Platforms. To be published.

DUNGAR R., ELDRED P.J.L. and SEVERN R.T. (1976) Dynamic Interaction of the Structure-Fluid-Foundation System of Gravity Platforms. Int. Conf. on Behaviour of Off-Shore Structures, Trondheim.

LYSMER J. and KUHLEMEYER R.L. (1969) Finite Dynamic Model for Infinite Media. J. Eng. Mech. Div. ASCE EM4, pp 859-877.

MALLARD D.J., IRVING J. and CORKERTON P.A. (1975) The Assessment of Seismic Design Criteria for Nuclear Power Stations in England and Wales. CSNI Specialists Meeting on Anti-Seismic Design for Nuclear Installations. Paris, December 1-3.

NEWMARK N.M. and ROSENBLUETH E. (1971) Fundamentals of Earthquake Engineering. Prentice-Hall.

WILSON E.L. (1965) Structural Analysis of Axi-symmetric Solids. A.I.A.A. Journal. Vol. 3 No. 12.

ZIENKIEWICZ O.C. (1971) The Finite Element Method in Engineering Science. McGraw Hill.

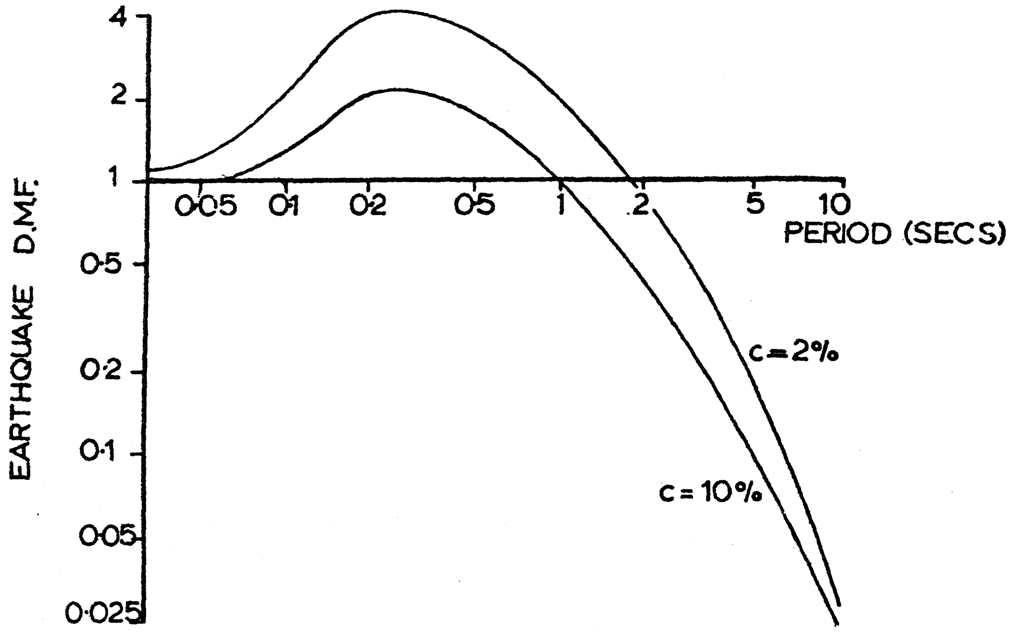

FIG. 1: EARTHQUAKE DYNAMIC MAGNIFICATION FACTORS

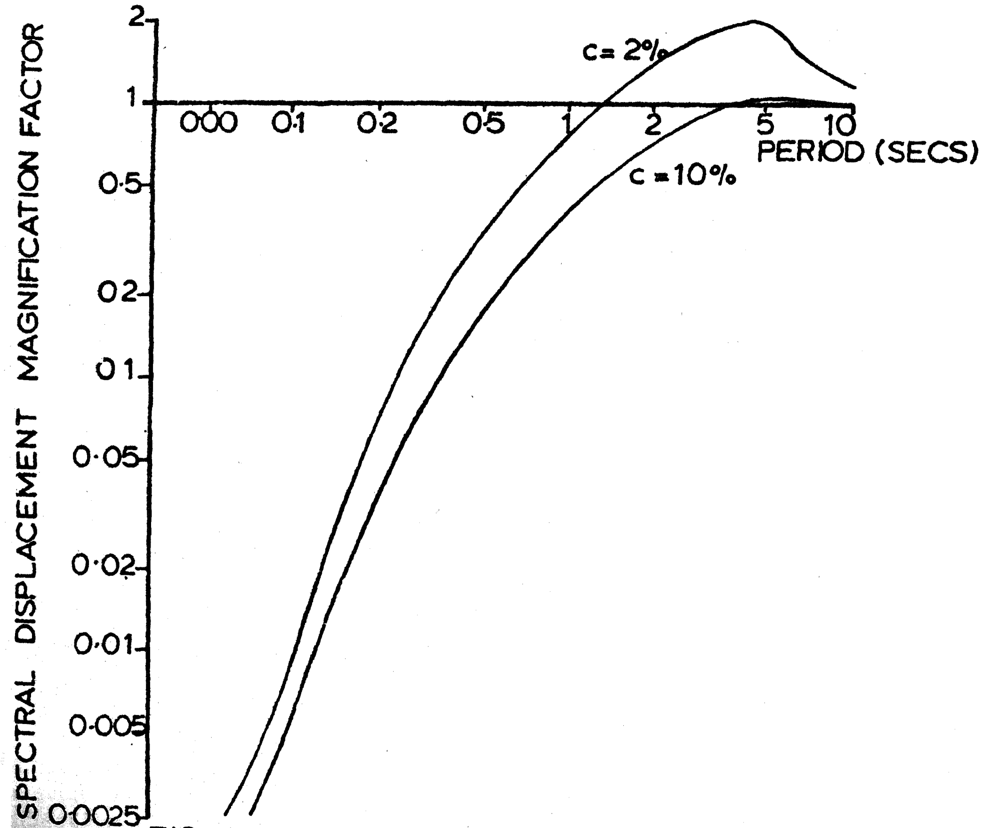

FIG. 2: SPECTRAL DISPLACEMENT MAGNIFICATION FACTORS

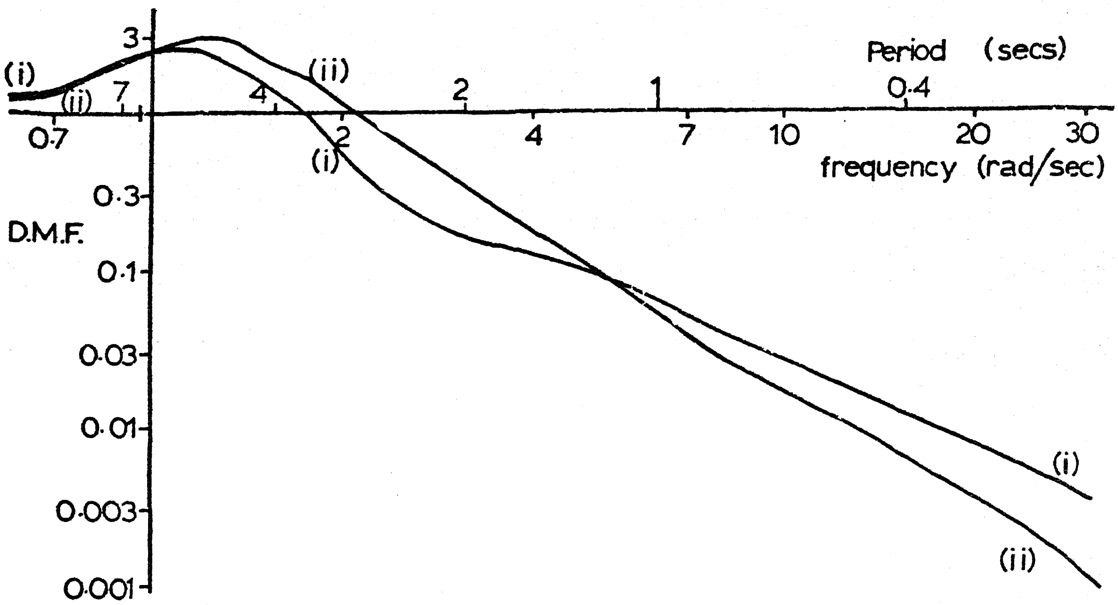

Fig. 3. MAGNIFICATION FACTORS FOR HOMOGENEOUS SOIL CONDITION.

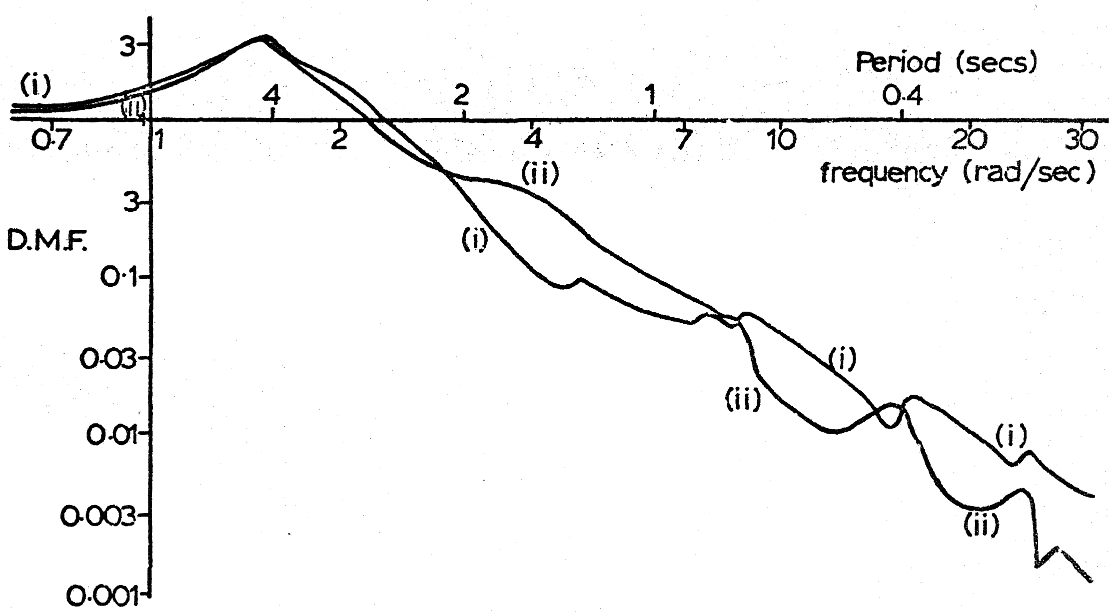

Fig. 4. MAGNIFICATION FACTORS FOR LAYERED SOIL.

DISCUSSION

S.L. Agarwal (India)

The use of gravity oil platforms, offshore oil activity in India has become very active in the Bombay High area, where platforms are supported on deep penetration piles. Soil layers change very frequently both in the vertical and horizontal directions. At any location, the soil in the top 100 ft. from the sea-bed, may consist of top 20 ft. very soft clay layer followed by four or five layers of calcareous sand, stiff clay, silty clay etc. The authors may indicate the modifications to their method to be applicable to the Bombay High area.

Author's Closure

Not received.