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DECONSTRUCTION OF STEEL PLATFORMS
CRUCIAL ENGINEERING ASPECTS

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INTRODUCTION

A number of different approaches have been considered for the deconstruction and subsequent disposal of steel jacket structures. The usual options include partial or complete removal, either 'piece-small' or in large sections, or toppling. The partial removal and toppling alternatives may be considered for platforms in deeper waters, whereas complete removal is the only option for decommissioning of platforms in shallow water.

There is now a limited but nonetheless growing body of experience of decommissioning platforms in deeper waters, initially in the Gulf of Mexico and more recently in the North Sea. Although many of the engineering aspects are still in the development stage, valuable lessons have been learned from recent projects.

In this paper, a number of the key aspects of the deconstruction of platforms in deeper waters are examined, and solutions to some of the major problem areas are put forward. The paper examines both practical design considerations, and the advanced numerical modelling which must be performed. These techniques are illustrated by reference to a design example.

DESIGN CRITERIA

The primary objective in the deconstruction of a platform is that the statutory requirement for free water above any remaining debris (the current consensus of opinion favours a value of 75 metres in the Northern North Sea) should be achieved. It is of vital importance that the deconstruction operation should be reliable, because it would be very expensive and probably also hazardous to deal with a partially failed structure. There is also little opportunity to take corrective action during a deconstruction operation, if events do not proceed as planned.

To fulfil these requirements, the following criteria should be adopted:

(1) Fail safe systems should be used where possible, so that the failure of any component will not result in the primary objective not being achieved.

(2) Critical systems should be duplicated.

(3) Untried technology should be avoided, unless absolutely necessary.

(4) It will not be possible to engineer all aspects of the work in fine detail, and so the engineering work should be concentrated on those aspects judged to be most critical.

The appropriate design philosophy entails a mixture of conventional engineering principles, such as might be employed in the design of permanent structures, and the more innovative approaches which are to be found in some demolition projects.

ENGINEERING ASPECTS

The decommissioning of steel platforms involves deconstruction followed by removal to a disposal site. The key question in planning the deconstruction is to decide upon the optimum number and size of sections to be removed - this is a trade off between the amount of underwater cutting to be performed and the size of the lifting tackle required (either in the form of lift barges or buoyancy tanks).

An option which is particularly attractive, for locations where it is permitted, is platform toppling. The first main advantage of this is that the deconstruction and disposal are combined into a single operation. Secondly, toppling is effectively a 'one piece' removal, but without the need for heavy lifting equipment. The remainder of this paper addresses the engineering aspects of platform toppling.

The structural engineering is carried out so that, by a combination of severing members, forming hinges and by applying destabilising forces, a toppling mechanism is formed which leads to the controlled failure of the structure. The design criteria for this are:

(1) The mechanism should be arranged so that, even if unexpected circumstances arise, the platform will come to rest on the seabed leaving the desired clearance above the structure.

(2) The formation of unpredictable failure mechanisms should be precluded at all costs.

(3) Underwater working should, to the extent practicable, be minimised.

(4) It is vital that parts of the structure which are required to remain intact during the toppling mechanism should not be damaged, for example by a temporary overload, or by the forces from explosive charges.

Several variants of platform toppling have been studied[1], of which a few have been put into effect on actual deconstruction projects. In one project, buoyancy tanks were used in the controlled toppling of steel jacket, and the remains of the platform were used to make an artificial reef[2]. Whilst this may have particular attractions for certain situations, it is not the preferred solution for a straightforward toppling operation since (a) the method does not comply with the 'fail safe' criterion (eg. the loss of a buoyancy tank would lead to an uncontrolled failure mechanism), and (b) there is considerable underwater working involved in the attachment and subsequent removal of the buoyancy tanks.

A more straightforward toppling operation undertaken in the Gulf of Mexico[3] in 1987 involved severing the piles at the mudline, and pulling the structure to failure using cables anchored to the seabed. The structure was an 8 pile jacket standing in 350' (75m) water depth, weighing a total of 3500 tons. The operation was executed according to plan, but it is interesting to note that the actual pulling force of 290 tons was 50% higher than predicted.

Whilst this latter scheme could be applied to North Sea structures, several modifications would be desirable. Firstly, many of the larger North Sea structures have a considerable number of piles (typically in excess of 20), and so it is better to cut the legs above the piles. Secondly, it is advantageous to arrange the failure mechanism so that the self weight of the structure contributes to the destabilising moment, both to reduce the required applied forces from the cables, and to create a more reliable mechanism. These features are realised in the toppling mechanism illustrated in Figure 1.

UNDERWATER CUTTING TECHNIQUES

In a platform deconstruction, there is a substantial amount of cutting of members to be performed underwater, and the use of explosives for this purpose is becoming an established technique. Some of the relative advantages and disadvantages of this method, viewed from the structural engineering standpoint, are discussed below.

A major advantage of the use of explosives for cutting is the reduced amount of underwater working, compared to mechanical or thermal cutting systems. Additionally, after the explosives have been placed and fused, all of the required cuts may be made simultaneously. With conventional systems, the members are cut one by one, with the cutting operation possibly continuing over an extended period. The structure is thus in a weakened condition for a longer period, and there is an increased possibility of storm damage occurring.

On the debit side, because the cuts occur simultaneously, there is no chance to take corrective action should the operation not proceed according to plan. One of the major potential risks is that a critical charge may not detonate, leading to an incomplete collapse of the structure. It is prudent to duplicate critical charges and their associated systems to counter this possibility.

The major problem associated with the explosive cutting is the potential for unwanted damage caused by the explosion, both to the environment and to parts of the structure which are required to remain intact during the toppling. Steps must therefore be taken to minimise this damage:

- undue conservatism should not be used in sizing the charges
- shaped charges should be used where possible for cutting steel members, since they are more efficient than bulk charges
- the charges should be placed as far away from critical members as possible
- consideration should be given to placing charges inside members - this gives the most effective use of the explosive, and partly contains the explosive forces
- measures should be taken to protect critical members, for example by internal grouting or by flooding the members concerned
- appropriate numerical simulations should be used during the engineering phase to investigate the effects of the explosive forces.

By using these techniques, it is possible to eliminate the uncertainty associated with explosive cutting, and to engineer reliable schemes for the deconstruction of steel platforms.

DESIGN STUDY - TOPPLING OF STEEL PLATFORM

Introduction

The considerations discussed in this paper will be illustrated with reference to a design study on the demolition of a steel riser platform. The platform forming the subject of this study was a 4 leg structure standing in deep water. This investigation considered toppling with the topsides in place. The wells and the conductors were assumed to be grout filled and to be demolished with the platform.

Selection of the Mechanism

To reduce the amount of steel to be cut in severing the legs, the legs could be cut just above the pile sleeves. The positioning of the cuts would leave the base of the structure intact. The most effective means of forming a mechanism was to remove a length from two adjacent legs, thereby forming a 'mouth', and letting the self weight of the structure act as a destabilising force.

Considerable interest centred on the braces at the level of the mouth (see member 'A' in Figure 1). Removal of these braces by explosive charges could have lead to uncertainty in the position of the hinges in the two remaining legs, and to an unpredictable failure mechanism. By leaving the brace members in place, the hinges would be constrained to form in the legs just above the pile guides. To assist in the formation of these hinges, light charges were considered at the hinge locations the objective being to weaken the joint cans.

The lower members were assumed to be flooded in order to resist radial pressure loads and to increase the overturning moment.

In studying the mechanics of the toppling, it was apparent that there would be relative movement between the conductors and their guide frames, and that this would have to be accounted for.

Behaviour of the Conductors

The behaviour of the conductors was deemed to have a potentially major influence on the toppling behaviour of the structure, and this was therefore studied in some detail. The conductors were chosen to be 30" in diameter and filled with grout, which represented a very significant additional strengthening of the platform. It was assumed that the conductors could prevent the toppling from taking place. It was calculated that it would not be possible to sever the conductors with reasonable quantities of explosives.

There were two basic mechanisms by which the conductors could prevent toppling, namely:

(1) Bending resistance.

(2) The development of axial forces.

Calculations indicated that the bending strength of the conductors was not a problem, since the effective lever arm was adequate to ensure the formation of plastic hinges. Regarding the possible axial load, it was computed that each conductor could support a force of approximately 600 tonnes between adjacent conductor guide frames. This would be sufficient to halt the platform toppling.

The said axial force could develop if the conductors were prevented from sliding freely through the guide frames. It was possible that the conductor connectors could catch on the guide cones.

A more detailed study revealed that if the conductors caught on the guide frames, the latter would fail without halting the platform toppling. However, if the guide frames failed, there would be nothing to pull the conductors over with the platform. It was therefore assumed that wire ropes should be placed around the platform at two levels, to confine the conductors and ensure that they would fail as intended.

Structural Analysis

A range of detailed analyses were performed to investigate the major aspects of the toppling. The principal analyses were:

(1) Determination of the rigid body motions.

(2) Structural analysis of the platform at various stages during the toppling.

(3) Analyses to determine the explosive loadings on the structure on the platform, and the local response.

The analysis of rigid body motions utilised a single degree of freedom model, in which the principal unknown was the angle of rotation of the structure (refer to Figure 2). The objectives of this analysis were to calculate time history plots, in order to generate inertial loads, drag forces and centripetal accelerations for inclusion in the main structural analyses. This analysis also produced time histories of the pivot forces, to enable the integrity of the hinges to be confirmed. The model accounted for the principal features of the system, ensuring that the results could be used with some confidence, and yet it was sufficiently simple to enable parametric studies to be performed with a minimum of computing effort. As an example of the outputs of the analysis, time histories of the angle of rotation and the reactions at the pivots for a typical set of governing conditions are shown in Figures 3 and 4. It may be seen that the toppling takes some 20 seconds to achieve a 60° angle of tilt (at this stage the toppling may virtually be regarded as complete, since the platform would inevitably proceed to ultimate failure).

The main structural analyses were performed for various 'snapshots' in time to determine the member forces and to confirm that critical members would not fail prematurely. In theory, these analyses should have been performed as transient, dynamic analyses; however, in practice, sufficient accuracy was obtained from quasi-static analyses in which a self equilibrating set of forces was applied to the structure. Several of these forces (eg. inertia and drag) were dependent on the platform motions, which were obtained from the 'rigid body' analysis described above. The structural analysis clearly demonstrated the major differences in the loadpaths through the structure during the toppling, but confirmed the adequacy of the basic structure to resist these loads.

The final set of analyses studied the effects of the explosive loading on members adjacent to the charges. The underwater explosion comprises an initial shock wave, leaving behind a bubble of hot gases which rapidly expands, but thereafter collapses re-compressing the gas which then may re-expand giving rise to a second bubble pulse. This cycle of bubble expansion/contraction may be repeated as many as seven times. Ultimately the internal energy will be dissipated and the gas will disperse.

The structure would be virtually transparent to the initial shock wave, owing to the short duration of the loading event, and the majority of the structural response results from the water particle motions caused by the gas bubble. By mathematical analysis, it is possible to compute the hydrodynamics, and hence to calculate the member loads from Morison's equation (see Appendix A). Typical time characteristics for inertia and drag loading are illustrated in Figures 5 and 6; it may be observed that the majority of the initial loading event has passed within 50 milliseconds of the explosion.

To determine the effects of the loading on the structure, it was sufficient to study the members closest to the charges, since the explosive forces decay rapidly with distance. Each member studied was modelled by a number of beam elements, and the effective loads, which varied both with time and with distance along the member, were applied to the model to perform a full transient dynamic analysis (see Figure 7).

Figure 8 shows the loading and response for Brace 'A' of figure 1. The loading trace shows that the loading follows closely the water particle acceleration as shown in figure 5, indicating that the inertia force dominates over drag. The peaks of loading correspond to times of minimum bubble radius when the bubble is re-bounding from the collapsed state. These pulses are repeated with decreasing intensity as the energy is dissipated. The bubble pulse period is about .35 seconds which is longer than the natural period of transverse vibration of the member, which is typically about .2 seconds. The response of the member is shown on the lower trace of figure 8. The initial impulse gives rise to a ringing response at the member natural period. The second pulse happens to occur at such a time as to reverse the motion and the third pulse gives the maximum displacement as it is reinforcing the response from the two previous pulses. The maximum displacement for this member at 10m from a charge was about 40cm assuming elastic deformation. It was found necessary to take into account tension effects and the formation of plastic hinges in order to produce a realistic prediction of the member's response. Grouting of this member was found to reduce the response by about 25%.

A further result obtained from the hydrodynamics study was the pressure transient resulting from the explosion. It was demonstrated that members in the vicinity of the charges would experience peak pressures which were significantly in excess of the hydrostatic pressure. The risk was that the pressure transient could trigger hydrostatic collapse, and it was decided that critical members should be flooded or grouted to avert this possibility.

CONCLUSIONS

In this paper the deconstruction of steel platforms has been examined, with particular reference to platform toppling. It has been shown that by a judicious combination of practical engineering and numerical studies, it is possible to engineer a practicable deconstruction technique. Toppling is seen to be an attractive solution to the deconstruction and disposal of steel platforms, for sites where it is applicable.

REFERENCES

(1) 'Final report for joint industry project. Platform abandonment', Wimpey Offshore report WOL 113/88, June 1988.

(2) Saunders DR et al., 'Toppling technique applied to platform removal in rigs to reef program', paper no. OTC 4761, Offshore Technology Conference, Houston 1984.

(3) Thornton WL and Quigel JC, ' Case history for rigs to reefs: a cost effective alternative for platform abandonment', paper no. OTC 5876, Offshore Technology Conference, Houston 1988.

APPENDIX A

HYDRODYNAMIC LOADS FROM UNDERWATER EXPLOSIONS

This appendix is a brief description of the method used to derive the hydrodynamic loads on cylindrical members from the oscillations of the gas bubble. It may be assumed that the shock wave which preceeds the bubble pulse has virtually no effect in exciting lateral movement of the members as their inertia is too great for the timescales of this loading (typically a few microseconds) even though about 60% of the energy of the explosion is propagated in this way.

As far as the surrounding water is concerned, the bubble may be modelled as an oscillating point source of mass with strength proportional to the rate of change of volume of the bubble. Consideration of equilibrium of the bubble surface allows the rate of change of bubble radius to be calculated from:-

$$\left(\frac{da}{dt}\right)^2 = \frac{2}{3}\frac{Po}{\rho}\left[\left(\frac{a_m}{a}\right)^3 - 1\right] \qquad \text{............(1)}$$

Where a is the instantaneous bubble radius
ρ is the density of the surrounding fluid
P_o is the ambient pressure at the charge position
and a_m is the maximum bubble radius.

The maximum bubble radius depends on the remaining energy E after the shock and the depth through the pressure P as below.

$$a_m^3 = \frac{3}{4\pi}\frac{E}{Po} \qquad \text{............(2)}$$

Assuming that the fluid is incompressible and that the flow is (locally) radial, the velocity potential 'ϕ' of the flow is given by:-

$$\phi = -\frac{1}{3r}\frac{d}{dt}(a^3) \qquad \text{............(3)}$$

from mass conservation. Here 'r' is the radial distance from the charge.

The water particle velocity v and acceleration a at any point in the fluid and at any time, may then be calculated from this equation by differentiation.

The normal force per unit length 'f' on a cylindrical member may then be calculated from the Morison equation as below:-

$$f = C_I\,\rho\left(\frac{\pi D^2}{4}\right)a_n + \frac{1}{2}D\,C_d\,\rho\,Vn|Vn| \qquad \text{.............(4)}$$

Where D is the member diameter

Ci is the inertia coefficient (taken as 2)

Cd is the drag coefficient (taken as 1)

a_n is the normal component of the water particle acceleration

v_n is the normal component of the water particle velocity

For subsequent pulses the energy E in equation 1 may be replaced by the energy available after the first bubble contraction. For multiple charges the field may be calculated from equation 3 where r is calculated for each charge and the time t reflects the time delay between charges. This assumes that the flow fields from each charge do not interact. The overpressure P may be calculated directly from the velocity potential using the linearised form of the Bernoulli equation. The dynamic pressure may be calculated from the water particle velocity v using the second order form of this same equation. In all practical cases the dynamic pressure may be neglected.

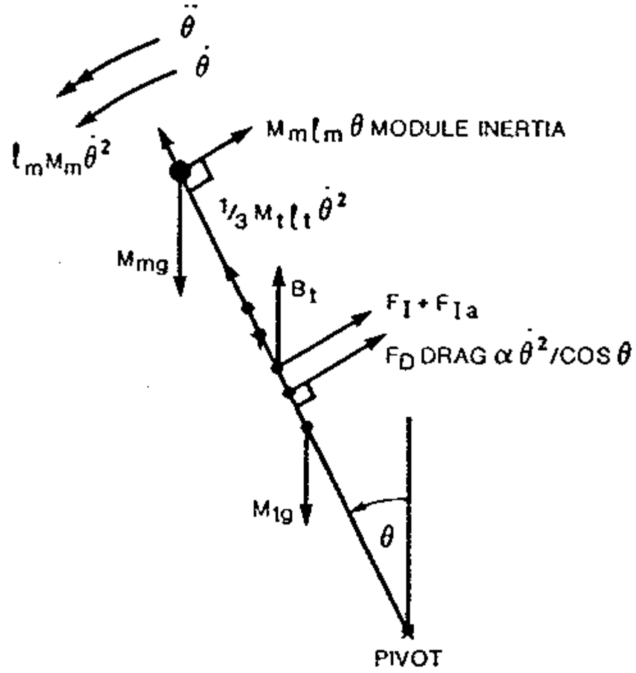

FIGURE 2

MODEL FOR ANALYSIS OF RIGID BODY MOTION

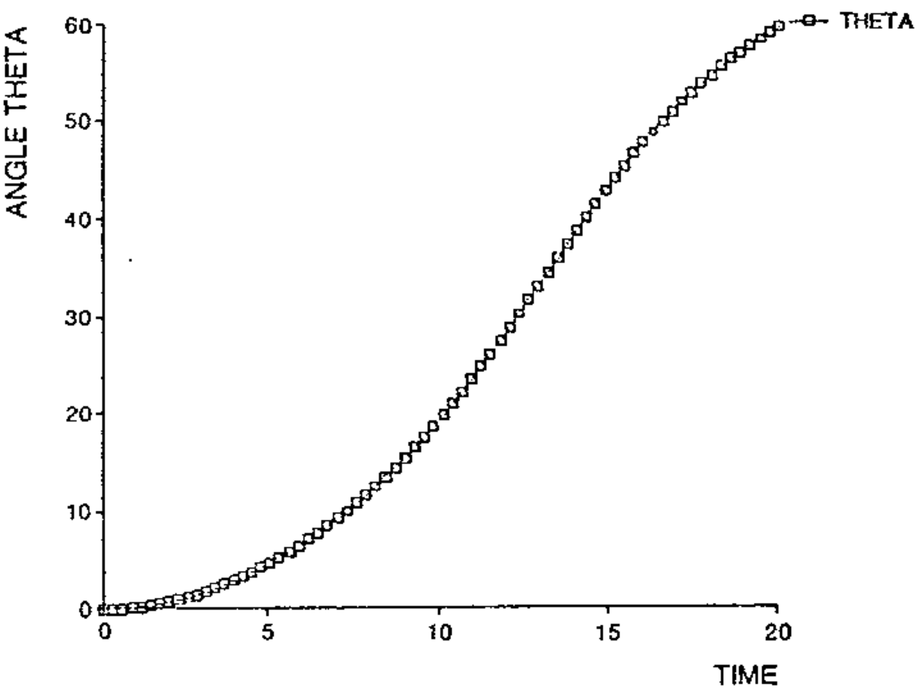

FIGURE 3

TIME HISTORY OF ANGLE OF TILT

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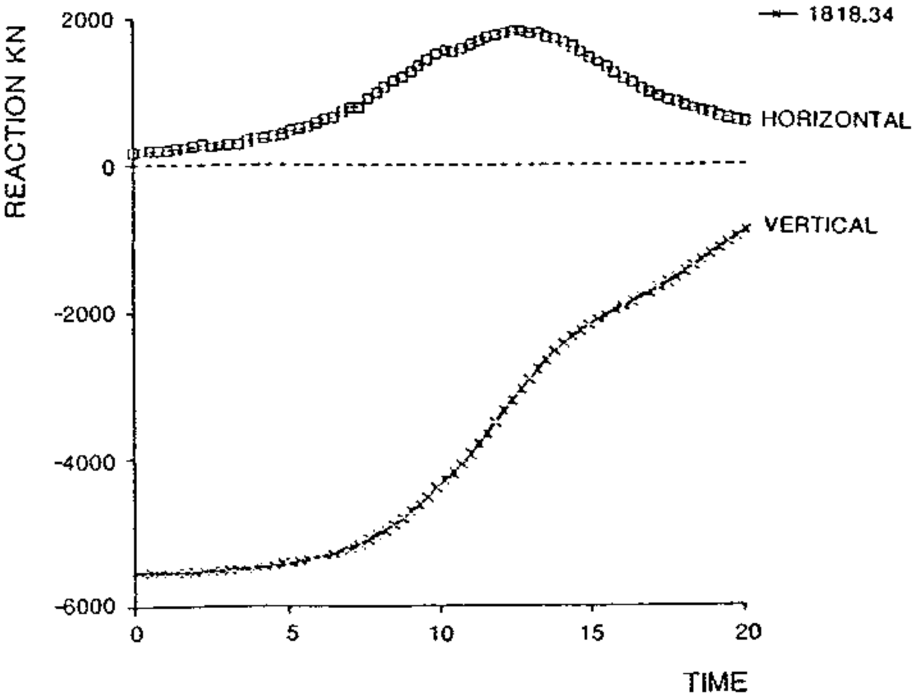

FIGURE 4

REACTIONS AT THE PIVOT

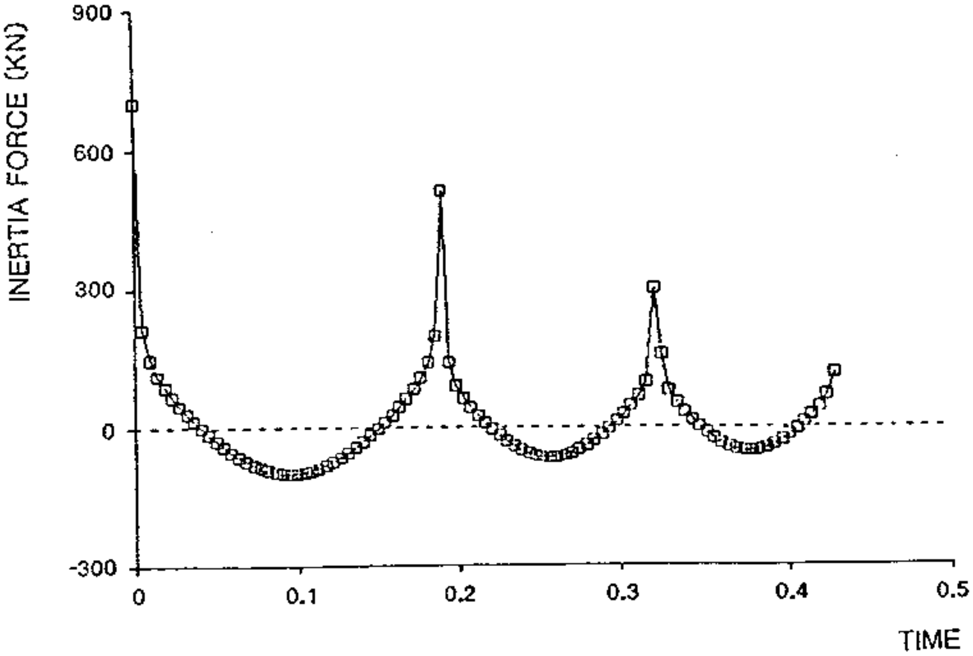

FIGURE 5

INERTIA FORCE ON CYLINDER

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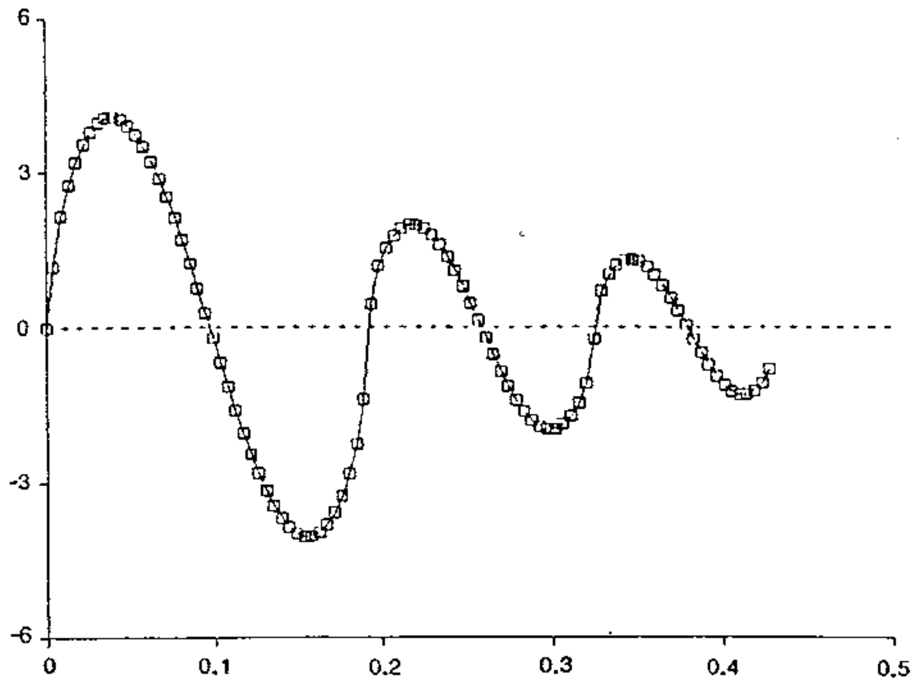


FIGURE 6

DRAG LOAD ON CYLINDER

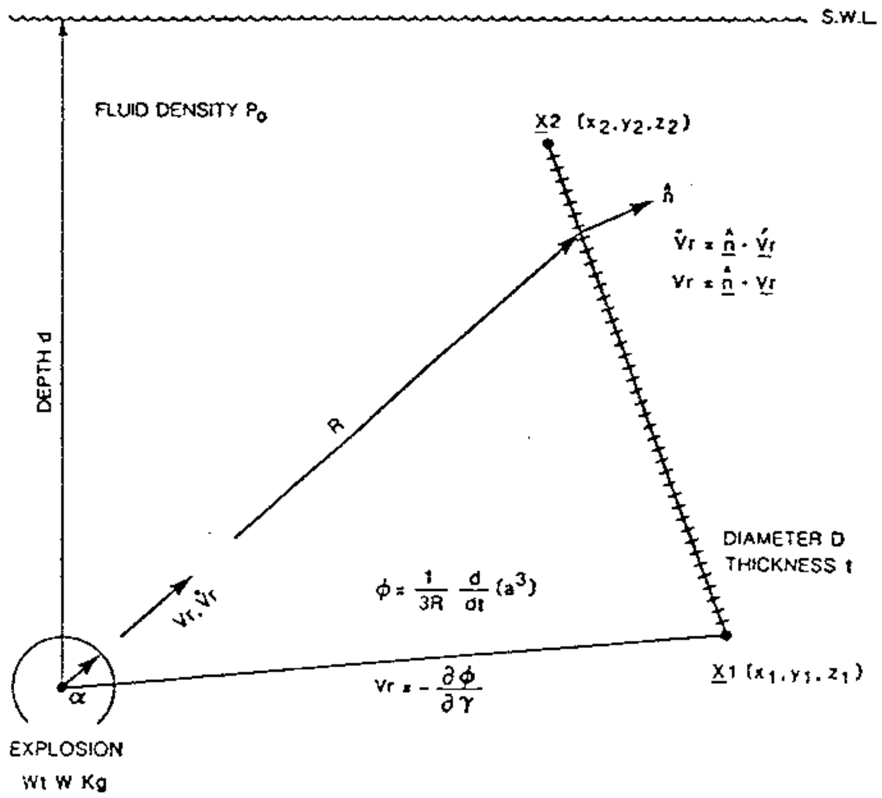
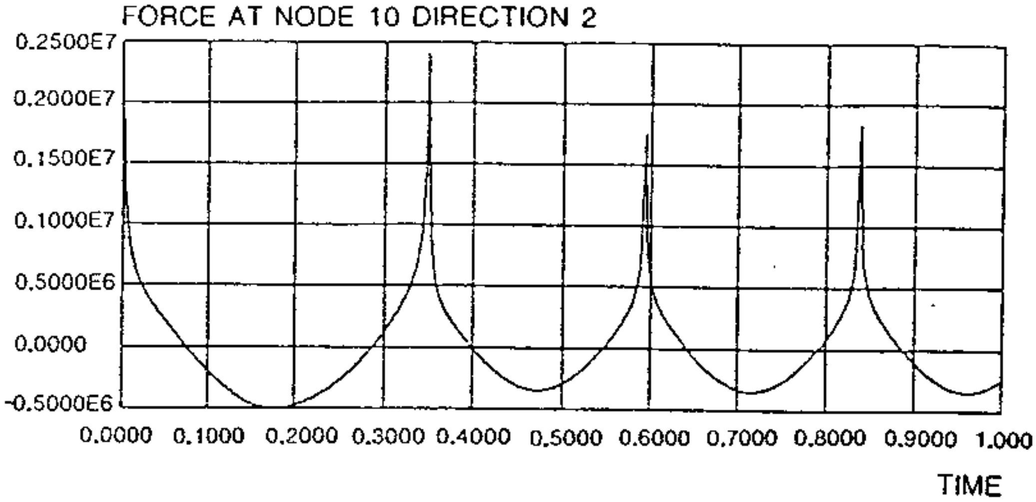
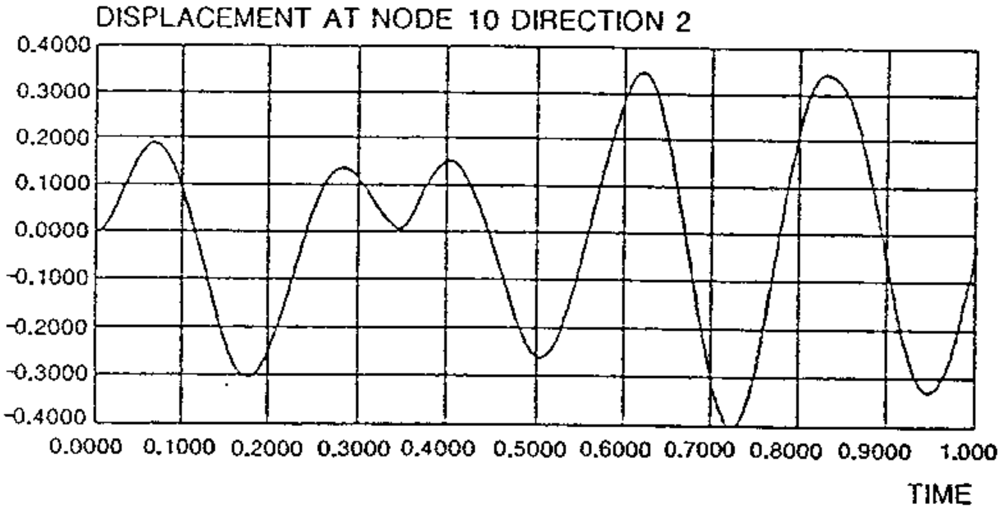


FIGURE 7

EXPLOSION LOADS ON CYLINDER - NOTATION



LOADING



RESPONSE

FIGURE 8

RESPONSE OF CYLINDRICAL MEMBER TO BUBBLE PULSE LOADING

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