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Artikel: On the modelling of sea-bed resistance for seismic response analysis of offshore pipelines in contact with the sea-bed

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On the modelling of sea-bed resistance for seismic response analysis of offshore pipelines in contact with the sea-bed.

Établissement de modèles de la résistance du fond de la mer en vue d'une analyse de la réaction sismique de pipelines sous-marins en contact avec le fond de la mer.

Über Modelle des Meeresbodenwiderstandes für die Analyse der seismischen Reaktion von Unterwasserrohrleitungen im Kontakt mit dem Meeresboden.

By

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SUMMARY

This paper contains a study of the seismic behaviour of marine pipelines in contact with the sea-bed, with particular reference to the mechanics at the contact region between the pipe and the sea-bed. Assuming seismic excitation to be transversely horizontal to the pipeline the problem has been solved by the finite element method for three different types of idealized sea-bed soil behaviour. Results show that pipe response depends upon the sea-bed resistance model and, as expected, the response of the segment in contact with the sea-bed is less than when there is no contact.

SOMMAIRE

Cet article contient une étude du comportement sismique de pipelines sous-marins en contact avec le fond de la mer et traite particulièrement la mécanique aux zones de contact entre pipeline et le fond de la mer. Le probleme a été traité par la méthode d'éléments finis pour trois types différents de comportements schématisés du sol marin en admettant que l'excitation sismique est transversalement horizontal au pipeline. Les résultats montrent que la réaction du pipeline dépend du modèle de résistance du sol marin. Comme attendu, la réaction d'un segment de pipeline en contact avec le fond de la mer est moindre que dans le cas où il n'ya pas de contact.

ZUSAMMENFASSUNG

Diese Abhandlung untersucht das seismische Verhalten von Unterwasserrohrleitungen im Kontakt mit dem Meeresboden, mit besonderer Bezichung auf die Mechanik an den Kontaktstellen zwischen Rohr und Meeresboden. Unter Voraussetzung einer zur Rohrleitung transversal-horizontalen seismischen Erregung wird das Problem durch Anwendung der Methode der endlichen Elementen für drei Verschiedene Arten von idealisierten Verhalten des Meeresbodens gelöst. Die Ergebnisse zeigen, dass die Rohrreaktion von Modell des Meeresbodenwiderstandes abhängt, und dass, wie zu erwarten, die Reaktion eines Rohrteiles im Kontakt mit dem Meeresboden kleiner ist als im Falle in dem ein solcher Kontakt nicht besteht.

1. INTRODUCTION

For various reasons pipelines are extensively used for gathering offshore oil and gas resources throughout the World including seismic regions. Usually these pipes are buried into jet-blasted channels in the sea-bed, although, when conditions do not permit this, they may be secured to the sea-bed in suitable lengths by means of concrete anchor blocks or screw piles. However, depending upon the composition of the sea-bed sediment and the prevailing marine conditions, a pipe segment which was initially buried may subsequently be exposed by the transportation of sediment away from the pipe location during periods of high scouring activity [1]; in a reverse of this process an initially exposed segment may also be totally or partially buried by the "duning" of the sea-bed. Clearly, an important design criterion here is therefore that the pipeline should be safe and stable under either of these two possible conditions.

In the context of offshore energy exploitation pipelines usually represent a substantial proportion of the total capital investment (in the Bombay High field, India, for example, a pipe network totalling over 140 km is envisaged). For this reason alone an accurate assessment of the safety and stability of offshore pipelines against seismic hazards is of paramount importance, particularly in regions where such hazards may be expected. Unfortunately, an accurate prediction of seismic behaviour with a view to formulating appropriate design criteria is usually a difficult proposition here, since system response in this problem is determined by the structural, hydrodynamical and soil mechanical aspects of the system, not to mention the complex dynamic interactions that may also take place between these aspects.

At the moment little published material on the dynamic/seismic behaviour of offshore pipelines appears to be available, understandably perhaps considering the relative infancy of this branch of technology. Over the years considerable research effort has been directed, on the other hand, to the solution of terrestrial pipeline problems including seismic response studies (references 2-9 contain a selection of research papers on the subject). Although it may be possible, prima facie, to extrapolate some of the findings/criteria of terrestrial pipelines to offshore pipelines, the fact remains however that current design practice relating to terrestrial pipelines is basically inadequate [5,10]; indeed, this inadequacy, as underlined by the aftermath of the San Fernando (1971, Richter magnitude 6.6) and the Managua (1972, Richter magnitude 6.25) Earthquakes, led to a series of recommendations by various prestigious committees [10,11]. Clearly therefore, a considerable research effort is still needed for formulating generally acceptable design criteria for terrestrial pipelines and, the need is even greater and perhaps more urgent in the case of offshore pipelines.

2. THE PROBLEM AND ITS ASPECTS

The problem to be investigated in this paper can be briefly stated as follows:

An offshore pipeline segment, supported between two anchor blocks (Fig.1a) is in contact with the sea-bed at one or more points or over a portion of its length. The segment is subjected to a transverse, horizontal and uniform seismic excitation which is transmitted to it, without dissipation, via the anchor blocks. To analyze the response of the segment with particular reference to the idealized sea-bed impedence parameters.

It would be instructive at this stage to focus attention onto the various aspects of the problem and also the assumptions which have to be made in order to construct a workable mathematical model of what is inherently a very complex system.

2.1 The structural aspect

Offshore pipelines are usually constructed of concrete coated enamelled steel pipe sections (Fig. 1b). The function of the enamel is to protect the steel pipe against external corrosion; the enamel, in turn, is protected by the concrete coating against accidental damage. As we have assumed seismic excitation to be transversely horizontal to the pipeline, it is clear that pipe response will be exclusively in the bending (in the Euler-Bernoulli sense) mode. It would be reasonable therefore to treat the segment as a beam in bending. However, as the concrete coating is likely to undergo progressive structural/chemical degradation with time [12,13], its contribution to the overall stiffness of the segment will be ignored. Furthermore, the pipe will be assumed to respond in a linearly elastic fashion ---- an assumption which is more likely than not to be valid in practice.

An important determinant of system response here is the nature of the end constraints of the pipe segment; a given segment will have greater response and pressure drag effects [12] with simply supported ends than with fixed ends. For this reason the test segments to be analyzed here will be assumed to have simply supported ends, although, in practice the end constraints are likely to lie somewhere between the fixed and simply supported conditions.

2.2 The hydrodynamical aspect

From the point of view of its physics the offshore pipeline problem belongs to a well-known class of coupled structure-fluid problems [14-18] in which the structural and fluid aspects interact, the extent of interaction depending upon the dynamic properties (e.g, natural frequencies, mode shapes, etc.) of the uncoupled aspects comprising the system. Systems, in which the structure is relatively flexible (as in pipeline systems), are negligibly affected by fluid compressibility; consequently the inviscid coupled behaviour of such systems can be studied merely by adding the so called added mass [14] to the structure mass in the system equation(s) of motion. If, on the other hand, non-linear pressure drag effect is significant, then the total hydrodynamic resistance to motion must be included in the system equation(s) in terms of both drag and fluid inertia forces. Following the Morison equation [19], for example, the drag and inertia forces, which are implicitly independent, can then be expressed in terms of appropriate drag (C_D) and inertia (C_M) coefficients. This is the usual approach in problems of this type and, in the case of offshore pipelines computed response is found to depend substantially on the value of C_M and to a much lesser extent on that of C_D [12].

Under conditions of potential flow the value of C_M can be shown [20-22] to decrease from a maximum of 2.29 for $d/D = 0$ (Fig. 1b) to an asymptotic minimum of 1.00 for d/D = infinity. For drag also experimental evidence indicates a significant correlation between C_D and the d/D ratio. In the case of a relatively smooth pipe for which $d/D > 1.5$, for example, Wilson and Caldwell [23] report $C_D = 1.7$ and 1.2 for Reynold's number equal to 33,200 and 56,600 respectively; under identical conditions but with $d/D <$

1.0 these C_D values were found to decrease by about 15%. Furthermore, both these parameters are likely to be significantly modified in practice by the structural/chemical degradation of the pipe coating [12] and also by the marine fouling on the pipe surface [12,24]. Unfortunately, published data relating to the in-situ values of these parameters do not appear to be available as of now. We will therefore use the classical (potential theory) values of C_M; in all cases the value of C_D will be taken as 1.5, which is probably realistic considering the complex flow condition around the pipe and also the possible environmental effects on the pipe surface.

Although the inertial and drag forces are assumed to be implicitly independent, in a complex separated flow such as this as yet little understood dynamic interaction(s) may take place between these forces [25,26]. Furthermore, in the presence of an ambient stream the pipe will undergo self induced (Strouhal) vibrations in the vertical plane; such vibrations may also lead, particularly at high Reynold's numbers, to a time-dependent wake and this may affect the pipe's drag and inertia coefficients relating to its transverse horizontal vibrations. A hydrodynamic coupling of this type between the horizontal and vertical vibrations of the pipe is a manifestation of the well-known "wake-body" interaction phenomenon [27,28]. As the mechanics of this complex phenomenon is not yet fully understood, we will ignore such interactions in this work.

For small values of the d/D ratio the pipe will also experience a significant lift force [20,21,23]. However, as we have ignored possible coupling between the horizontal and vertical vibrations of the pipe and as we are interested only in its transverse horizontal response, this lift force is no longer relevant here.

2.3 The soil-mechanical aspect

Considering the complex behaviour of sea-bed soil including possible thixotropy and soil-liquefaction effects under dynamic/cyclic conditions of loading, it is clearly difficult to devise a general mathematical model to represent accurately the mechanics at the pipe-seabed interface. Here again the inadequacy of published data relating to the in-situ behaviour and properties of sea-bed soil seriously inhibits any attempt at realistic model studies. We will nevertheless examine a number of models based on idealized soil behaviour; clearly, their practical validity would depend very much on the extent to which in-situ behaviour corresponds with idealized behaviour.

We will assume that the resistance offered by the sea-bed against pipe motion can be idealized in the elasto-plastic sense by means of a kinetic coefficient of friction (μ) between the pipe and the sea-bed, and parameter Q which denotes the maximum elastic ground displacement. For simplicity both μ and Q will be assumed to be constant although in practice they are both likely to vary, particularly with time.

The pipeline problem with ground contact considered here is basically one of pipe-seabed interaction. Figs.2a and b show the deformed pipe geometry with a single central contact point and the forces active at that point. For conceptual simplicity if the pipe is now represented by a linear elastic spring of stiffness k_p and the sea-bed by an elasto-plastic spring of stiffness k_s (which is a function of displacement), then it is clear that the coupled response of the pipe will be a function of both k_p and k_s. It

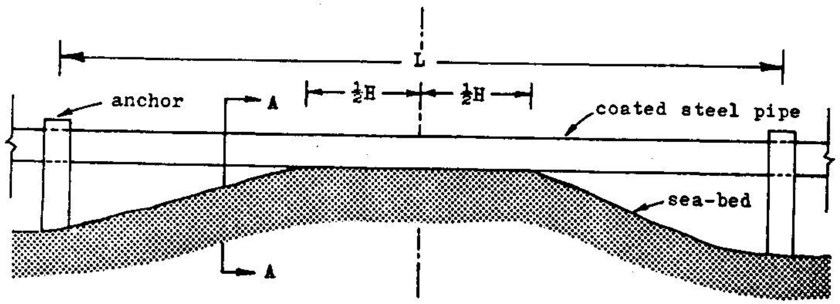

FIG. 1a Pipe-seabed configuration under consideration.

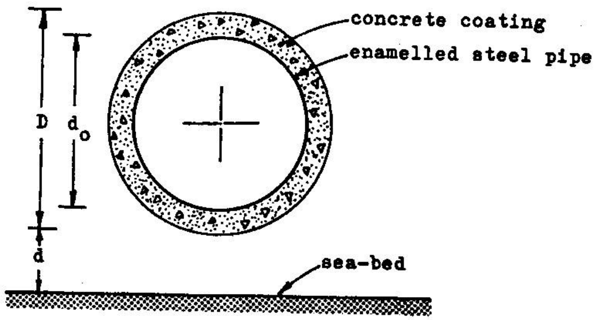

FIG. 1b Magnified pipe section at A-A in Fig. 1a.

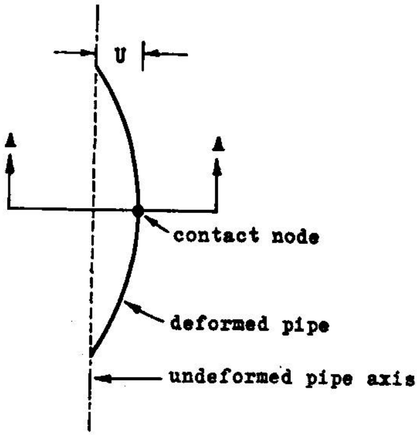

FIG. 2a Deformed pipe geometry with a single central contact node.

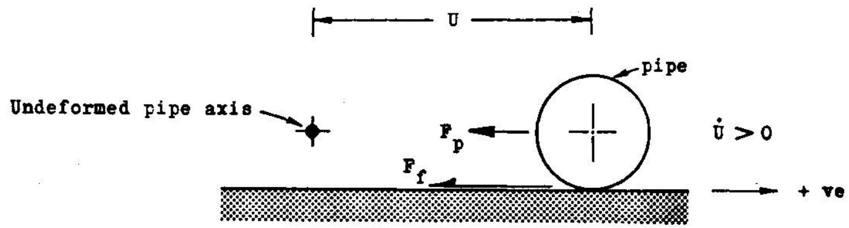

FIG. 2b Magnified section at A-A in Fig. 2a showing forces at the contact node.

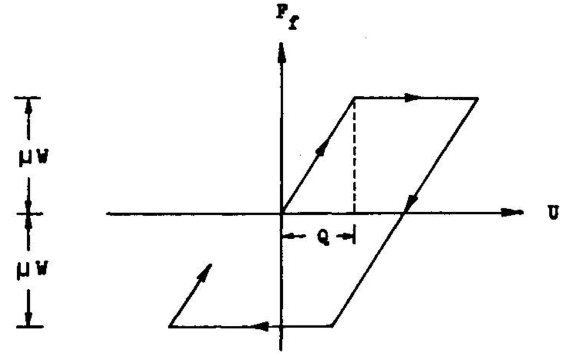

FIG. 2c Sea-bed resistance model "A".

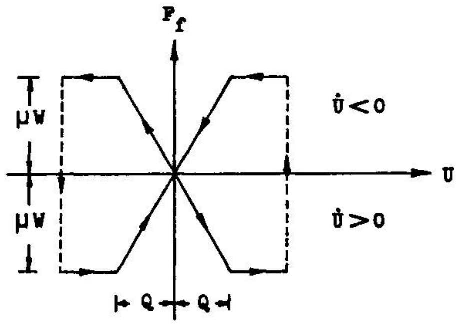

FIG. 2d Sea-bed resistance model "B".

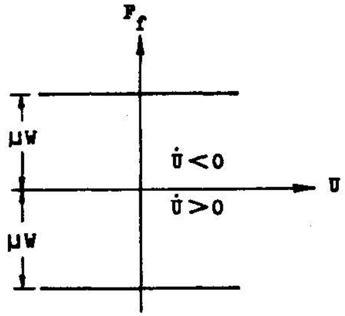

FIG. 2e Sea-bed resistance model "C".

is also clear from the mechanics of the coupled system that these springs act in parallel. Consequently, when k_s dominates, i.e, $k_s >> k_p$, the pipe is likely to undergo irrecoverable deflections caused by large plastic forces that will now be mobilized by the sea-bed. In this situation the sea-bed resistance model (model A), shown in Fig. 2c, is likely to be valid. If, on the other hand, $k_p >> k_s$, then the pipe will not undergo significant irrecoverable deflections, and consequently, the sea-bed resistance model (model B), shown in Fig. 2d, is more likely to be valid. In another model (model C), shown in Fig. 2e, which is also worth considering, the contact point(s) is assumed to move only when the force in the pipe ($|F_p|$ in Fig. 2b) becomes equal to μW. The validity of this model is difficult to justify, except perhaps when the peculiarity of the sea-bed (caused, for instance, by the presence of rocky obstacles or displaced sand-bags initially placed upon the pipe in order to increase its stability) constrains the pipe to move in this way.

3. THEORY

Following a spatial discretization of the pipe segment consisting of (n-1) finite elements interconnected by n nodes of which m nodes ($m < n$) represent the pipe-seabed interface, the implicit equation of motion of the segment can be written with reference to Fig. 2b as [29]

$$\{F_p\} + \{F_f\} \operatorname{sgn} \{\dot{U}\} = \{0\} \tag{1}$$

in which, considering seismic excitation only, we can show that [12,17,19]

$$\{F_p\} = [K]\{U\} + [C]\{\dot{U}\} + [M + M_a]\{\ddot{U}_t\} + \tfrac{1}{2}\rho C_D[A]\{\dot{U}_t|\dot{U}_t|\} \tag{2}$$

Here vector $\{F_f\}$ lists the sea-bed resistances concentrated at the m contact nodes. [K], [M] and [C] denote stiffness, submerged mass and viscous damping matrices of the segment, respectively, while $[M_a]$ denotes its added mass matrix (both [M] and $[M_a]$ are diagonal matrices). $\{U\}$ lists pipe deflections relative to the moving undeformed pipe axis while $\{U_t\}$ lists total pipe deflections from a fixed reference. Dots denote differentiation with respect to time, ρ the mass density of water and [A] the diagonal matrix of pipe areas projected along the direction of motion. Then, substituting

$$\{U_t\} = \{U_g\} + \{U\} \tag{3}$$

into Eq.(2), where $\{U_g\}$ lists ground displacement history, we now obtain from Eqs.(1) and (2)

$$[K]\{U\} + [C]\{\dot{U}\} + [M + M_a]\{\ddot{U}\} + \{F_f\} \operatorname{sgn} \{\dot{U}\}$$
$$+ \tfrac{1}{2}\rho C_D[A]\{(\dot{U} + \dot{U}_g)|(\dot{U} + \dot{U}_g)|\} = - [M + M_a]\{\ddot{U}_g\} \tag{4}$$

The response of the pipe to a given ground excitation record will now be found by solving Eq.(4) with appropriate initial and boundary conditions.

4. SOLUTION DETAILS

The pipe segment, supported between two anchor blocks (Fig. 1a) was represented by 6 finite beam elements and its rotational degree-of-freedom was condensed-out in order to minimize the size of matrices to be processed. Structural damping of the pipe was taken as 5% of critical which appears to be usual in slender offshore structures of this type [17]. The thickness of the concrete coating was determined by requiring that the total submerged weight of the empty segment, including the coating, be 10% greater than its buoyancy so that the empty pipe was prevented from floating upto the surface. The stiffness of the coating was however ignored for reasons given in section 2.1. At all times the pipe was assumed to be completely filled with oil (mass density taken as 1.82 lb/cu. ft.) as its maximum response occurred in this condition.

A numerical algorithm was implemented for the solution of Eq.(4); in this the mass of the pipe was represented by nodal lumped masses while the time domain was discretized in the finite (central) difference sense. The processing in the time domain by this device basically amounts to an explicit forward integration procedure [30] in which pipe response at time $(t + \Delta t)$ is calculated on the basis of already computed (or prescribed) responses at times t and $(t - \Delta t)$. An iterative loop was included in each time-step to deal with the non-linear pressure drag term in Eq.(4); convergence of the iterated solution was found to be rapid and a sufficiently converged solution was obtained with less than 6 iterations per step. The size of the time-step (Δt) to be used was optimized with respect to the convergence and stability of solution. The high degree of accuracy that this algorithm is capable of has been demonstrated in reference 12.

The nodal contact reactions, $\{W\}$, between the pipe and the sea-bed will obviously depend on the end-constraints of the segment and also on the elevations or settlements of the contact nodes with respect to the end supports. For the sake of simplicity $\{W\}$ was calculated in this study by assuming the segment to be resting on a total of $(m + 2)$ supports, all at the same level (m denotes the number of contact nodes).

For a given sea-bed resistance model the value of the ground resistance vector $\{F_f\}$ to be used in a given time-step in Eq.(4) was found from the idealized plot of that model (Figs. 2c - 2e).

5. RESULTS AND DISCUSSION

The seismic response of a test pipe segment (details given in table 1) to the Taft earthquake of 1952 was calculated relative to the three sea-bed resistance models shown in Figs. 2c-2e. Table 1 contains a summary of the maximum (beam) bending moment response of the segment for various parametric combinations. Clearly, the overall effect of ground contact is to diminish response compared with the no-contact situation, as we might have anticipated and, the amount of response reduction depends on the type of sea-bed resistance model implemented.

In an elasto-plastic idealization of sea-bed behaviour, it is clear that the maximum sea-bed resistance retarding pipe motion will be mobilized in the plastic zone (whose threshold is defined by the parameters Q and μ). Consequently, response attenuation due to contact will be expected to be greater in the plastic zone than in the elastic zone. Therefore, for a given Q an increase in the value of μ would diminish plastic response; consequently, at a given relative sea-bed stiffness (K_r) response will be ex-

Table 1 Peak bending moment response of a simply supported pipe segment to the Taft earthquake of 1952.

H/L	Q/L	μ	Peak bending moment/10^4 (lbf-ft)		
			Model A	Model B	Model C*
0**		0	36.00**		
0.167	0.00200	0.025	35.96	35.10	28.80
0.167	0.00200	0.050	35.82	34.92	7.20
0.167	0.00200	0.075	35.78	34.94	6.84
0.167	0.00200	0.100	35.71	35.10	7.92
0.167	0.00100	0.025	34.50	34.33	
0.167	0.00100	0.050	34.81	34.26	
0.167	0.00100	0.075	34.42	34.21	
0.167	0.00100	0.100	33.14	33.22	
0.167	0.00067	0.025	33.45	32.69	
0.167	0.00067	0.050	32.93	30.25	
0.167	0.00067	0.075	31.30	30.30	
0.167	0.00067	0.100	35.18	30.60	
0.500	0.00200	0.050	37.91	35.76	
0.500	0.00100	0.050	36.78	34.00	
0.500	0.00067	0.050	35.00	28.12	

* The ratio Q/L is not relevant in this model; ** no contact between the pipe and the sea-bed and consequently, this value is not dependent upon any model.

System details: The pipe segment is simply supported between two anchor blocks. L = 100.00 ft., D = 3.65 ft., d_o = 3.00 ft., EI = 1.85 x 10^9 lbf-ft, structural damping = 5% of critical; C_D = 1.5, C_M = 2.3 for the contact zone and 1.5 elsewhere.

pected to increase with increasing values of μ, since this would amount to increasing the size of the elastic zone. Secondly, and for the same reason, an increase in the value of Q will be expected to lead to increased response for a given μ, since clearly an increased Q leads to an enlarged elastic zone.

Both the above aspects have been vindicated [29] in the case of model B. The second observation is also valid in the case of model A, as may be seen from Table 1; however, the first observation does not strictly apply to this model, as may be seen from Fig. 3.

It will be seen from Table 1 that response attenuation due to contact is maximum in the case of model C. It is interesting to observe that in this case response attenuation is very steep within the range $\mu = 0 - 0.04$ and, for $\mu > 0.04$ response increases slowly with increasing values of μ, as can be seen from Fig. 4. This increase may be explained as follows: according to model C the contact node(s) will move relative to the sea-bed only when $|F_p| = \mu W$ ($|F_p|$ cannot be greater than μW); otherwise the contact node(s) remains stationary relative to the sea-bed. Therefore, as long as $|F_p| \neq \mu W$, energy will be stored up in the segment due to the motion of the part of the segment not in contact with the sea-bed, and clearly, the amount of stored energy will be directly proportional to the value of μ. Then, when the condition $|F_p| = \mu W$ is fulfilled, the hitherto stationary contact node(s) will be jerked into motion causing the pipe to respond slightly more, depending on the value of μ, than it would have done with a smaller value of μ.

The effect of increasing the contact zone length on response at various values of the friction coefficient is worth noting (Table 1).

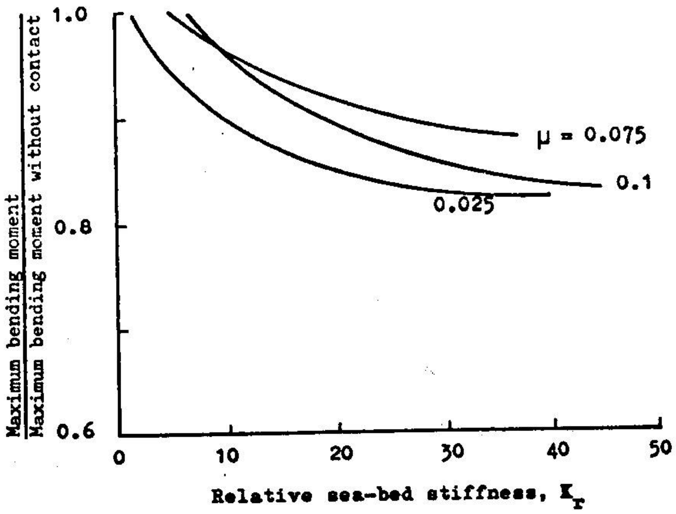

FIG. 3 Variation of the maximum bending moment response of the segment with relative sea-bed stiffness (model A).

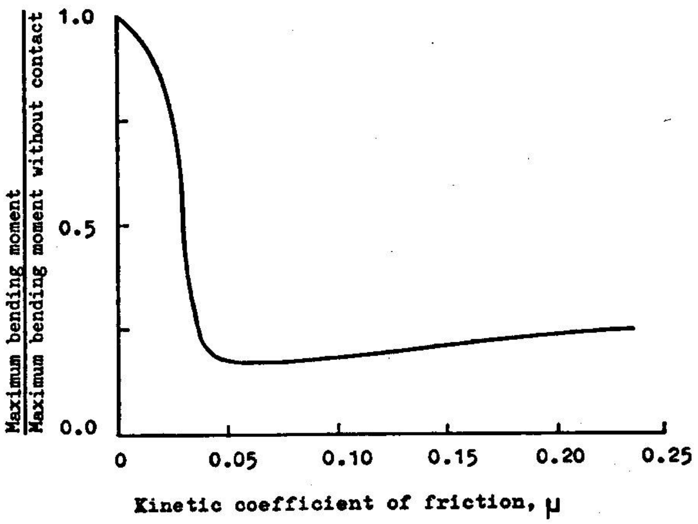

FIG. 4 Variation of the maximum bending moment response of the segment with the kinetic coefficient of friction (model C).

6. CONCLUSIONS

A computer coding has been developed for the seismic/dynamic analysis of offshore pipelines. This coding was implemented in this study to calculate the bending response of a typical offshore pipe segment, which is partially in contact with the sea-bed, to the Taft earthquake of 1952. Analysis was made by assuming ground excitation to be transversely horizontal to the pipe axis. Results show that for a given segment in a given hydrodynamic environment, pipe response is determined by the soil-mechanical behaviour of the sea-bed at the contact zone and the length of this zone. Testing of the system with three different idealized sea-bed soil resistance models showed that the overall effect of ground contact is to reduce response compared with the no-contact situation, as expected, the amount of reduction depending upon the model and the parameters defining it.

Despite the limited scope of this study two observations emerge from it and these may have important bearing on the design of such pipelines: firstly, models A and B are more likely to be valid in practice than model C, and, in A and B the response attenuation due to partial contact is not substantial over a realistic range of idealized sea-bed parameters. It would follow, therefore, that a design based on the no-contact assumption would not necessarily lead to a seismic over-design. Secondly, it is clear that in the interest of seismic safety against bending failure the amount of contact between the pipe and the sea-bed should be maximized. The buried pipe obviously represents the ideal situation in this respect. It would be prudent therefore to carry out a site survey with particular reference to scouring of the sea-bed; if high scouring activity is indicated then appropriate measures (e.g, sand-bagging, screw-piling, etc.) should be taken against it so that an initially buried segment is not subsequently exposed by sediment transport.

This study should be regarded as essentially a first step towards the understanding and solution of a very complex but important problem. A considerable amount of investigative work still remains to be done, particularly in the hydrodynamical and soil-mechanical aspects of the problem.

7. NOTATION

[A]	Diagonal matrix of projected pipe areas.
[C]	Viscous damping matrix of pipe.
C_D	Drag coefficient = 2*drag force/(ρ*projected area*velocity^2)
C_M	Inertia coefficient = added mass of pipe per unit length/mass of water displaced by pipe per unit length.
D	External diameter of pipe including concrete coating.
d	Clearance between the pipe and the sea-bed.
d_o	External diameter of pipe excluding concrete coating.
E	Young's modulus of steel pipe.
$\{F_f\}$	Listing of sea-bed resistance forces at the contact nodes (F_f refers to a single contact node).
$\{F_p\}$	Listing of restoring forces at the pipe nodes (F_p refers to a single node).
H	Length of contact zone.
I	Second moment of area of pipe.
[K]	Stiffness matrix of pipe.
K_r	Relative sea-bed stiffness = $(\mu W/Q)/(EI/L^3)$
L	Span of pipe segment
[M]	Submerged mass matrix of pipe.
$[M_a]$	Added mass matrix of pipe.
Q, μ	Idealized sea-bed soil resistance parameters (Figs. 2c - 2e).
{U}	Listing of pipe deflections from the moving undeformed pipe axis.
$\{U_t\}$	Listing of pipe deflections from a fixed reference.
$\{U_g\}$	Listing of sea-bed displacement from a fixed reference.
{W}	Listing of nodal reactions between the pipe and the sea-bed(W refers to a single node).
ρ	Mass density of sea-water.

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