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WAVE FORCES ON A PIPELINE IN A TRENCH

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ABSTRACT

Most existing design information for wave forces on pipelines deals with the case of pipes on a flat bottom, not in a trench. However, the Beaufort Sea environment will require submarine pipelines to be placed in trenches. Therefore, a series of tests representative of Beaufort design conditions was conducted at a scale of 1:15 in the wind-wave flume of the NWRI Hydraulics Laboratory. The drag and lift coefficients in the Morison equation, calculated from the measured forces, were found to be comparable to values reported in the literature for pipes on flat beds at the same total depth of water. Flow visualization tests conducted at scale of 1:60 in a smaller flume supported this conclusion.

RESUME

La plupart des données théoriques existantes sur les forces exercées par les vagues sur les pipelines traitent du cas de tuyaux sur un fond plat, et non dans une tranchée. Cependant, l'environnement de la mer de Beaufort nécessitera l'installation de pipelines dans des tranchées. Une série d'essais représentatifs des conditions théoriques dans la mer de Beaufort a donc été menée à l'échelle de 1/15 dans le bassin à houle due au vent du Laboratoire d'hydraulique de l'INRE. Les coefficients de traînée et de soulèvement dans l'équation de Morison, calculés à partir des forces mesurées, se sont avérés comparables aux valeurs indiquées dans la documentation pour des tuyaux sur fond plat à une même profondeur d'eau. Des essais de visualisation des écoulements à l'échelle de 1/60 dans un bassin plus petit corroborent cette conclusion.

MANAGEMENT PERSPECTIVE

Suitable estimations of the design conditions for construction of submarine pipelines in the Canadian Arctic are vital to minimize the chance of an oil spill. Proposed pipelines in the Beaufort Sea will likely be laid in trenches to avoid scour due to ice floes. The model tests reported here indicate that the forces due to waves on a pipeline in a trench can be approximated by forces on a pipe on a flat bottom at the same total depth. This result means that the design for wave forces for pipelines in trenches (for which there is no available information in the open literature) can make use of the extensive body of literature available for pipelines on flat bottoms.

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PERSPECTIVE-GESTION

Il est essentiel de pouvoir compter sur des évaluations appropriées des conditions de calcul pour la construction de pipelines sous-marins dans l'Arctique canadien afin de minimiser les risques d'un déversement. Les pipelines proposés pour la mer de Beaufort passeront probablement dans des tranchées afin qu'ils soient à l'abri du décapage par les floes. Les essais sur modèle dont il est question ici indiquent que les forces attribuables aux vagues qui s'exerceront sur un pipeline déposé dans une tranchée peuvent être assimilées aux forces qui s'exercent sur un pipeline posé sur le fond à la même profondeur totale. Cela signifie que le calcul des forces exercées par les vagues sur les pipelines déposés dans des tranchées peut être fondé sur l'importante documentation relative aux pipelines installés directement sur le fond; il n'existe pas de renseignements relatifs aux pipelines en tranchées dans la littérature publique.

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1.0 INTRODUCTION

Exploration activity for oil and gas in the offshore regions of Canada and subsequent planning for offshore production facilities have increased the interest in the design of submarine pipelines. In the Beaufort Sea, submarine piplines are vulnerable to damage from keels of ice floes (Hnatiuk and Wright 1983). To avoid this problem it has been proposed to place the pipelines in trenches below the maximum anticipated depth of scour. Consequently, there is a need to address the problem of wave and current loadings on pipelines in trenches.

Over the last several decades there has been a great deal of research on the subject of hydrodynamic loadings on submarine pipes. Almost all of the work in the open literature, however, has focused on pipes on flat beds. As reported by Atken (1982), and Bryndum et al. (1983), most analyses of forces on pipelines are based on the semi-empirical equations to predict forces in oscillatory flow derived by Morison et al. (1950). There has been a very large scatter in the results reported, and it is only recently that experimental techniques and methods of analysis have been developed sufficiently so that results show a clear dependency of the coefficients of drag, inertia and lift on the nondimensional variables of the problem. Sarpkaya and his co-workers (see Sarpkaya and Isaacson 1981), using a water tunnel, were among the first to show such a clear dependence. Bryndum et al. (1983), using a pipe-seabed model mounted on an oscillating carriage in a flume, also produced results of high quality. Littlejohns and Spencer (1982) presented high quality field data. A review of these studies and others can be found in Skafel (1985). Virtually no attention has been given to the question of wave forces on pipes in trenches. A recent paper by Sumer et al. (1989) does discuss the dynamic response of pipes in scour trenches. In their work, however, the trench depth is approximately the same size as the pipe diameter, quite a different configuration to that tested in this study, where the pipe lies at the bottom of a trench whose depth is an order of magnitude greater than the pipe diameter.

This study was undertaken to explore the question of pipelines in trenches using the large wave flume at Canada's National Water Research Institute. After discussions with oil company officials, the pipe-trench configurations to be tested were selected to represent those which would be typical of Beaufort Sea applications. In addition, flow visualization tests were conducted in a small flume to gain insight into the complexities of the flow close to the pipe.

2.0 METHOD OF ANALYSIS

Typically, estimation of forces on pipelines on or near flat beds has been done using the Morison equation (Morison et al. 1950), which was developed originally for piles. This equation uses the concept of a drag force and an inertial force collinear with the flow, and a lift force normal to the flow, and corresponding coefficients. Application of this approach to the oscillatory flow regime under waves

has not proved to be simple. There are many examples in the literature where the drag and inertia coefficients show considerable scatter when plotted as a function of the relevant dimensionless variables. Recently, a new approach has been developed in which the coefficients are time dependent and the velocity is modified to include the pipe's encounter with its own wake (Verley et al. 1989). Nevertheless, the Morison equation continues to be used, and is reviewed briefly below. (For an excellent overview of the whole question of wave loading on structures, see Sarpkaya and Isaacson 1981).

The assumption behind the Morison equation is that the instantaneous force on the pipe can be represented by the sum of a term proportional to the square of the velocity and a term proportional to the acceleration. For the horizontal force it takes the form:

$$[1] \qquad F_h = 0.5\, C_d \rho D U |U| + 0.25\, C_m \pi \rho D^2 A$$

where F_h is the horizontal force, ρ is the water density, D is the pipe diameter, U is the instantaneous horizontal velocity, A is the corresponding water acceleration, and C_d and C_m are the drag and inertia coefficients. The lift force on the pipe due to the horizontal velocity is given by:

$$[2] \qquad F_v = 0.5\, C_v \rho D U^2$$

where F_v is the vertical force, and C_v is the lift coefficient.

Dimensional analysis (Garrison 1980; Sarpkaya and Rajabi 1979) shows that the drag and inertia coefficients can be expressed as functions of the non-dimensional parameters

$$2\pi t/T,\ UT/D,\ D^2/T\nu,\ k/D,$$

where t is time, T the wave period, ν the kinematic viscosity, and k the roughness of the pipe. It is common practice to consider the coefficients independent of phase (because of the method of analysis). The second parameter is the Keulegan Carpenter number (K), the third has been referred to as the frequency parameter (β), and the last is the relative roughness of the pipe. The Reynolds number (Re) is the product of K times β. There are several ways of estimating the coefficients, the simplest being to select the force at the appropriate phase of the time series, measure or calculate the velocity and acceleration, and compute the coefficients. More sophisticated techniques include the Fourier averaging method and the method of least squares. The latter (Littlejohns and Spencer 1982), particularly suitable for analysis by computer, was used in the present study and is described in Skafel and Bishop 1990.

In the tests reported here, velocity measurements were not made. In their place, the waves were measured directly over the pipe, and the corresponding velocity and acceleration time series were computed via Fourier analysis of the surface displacement, using second order Stokes wave theory. The wave velocities and accelerations were calculated for the depth of the centreline of the pipe, as placed in the trench.

3.0 EXPERIMENTAL SETUP AND PROCEDURES

The experiments to measure wave-induced forces on pipelines were conducted in the large wind-wave flume in the Hydraulics Laboratory, NWRI. In order to create a trench, a mortar-veneered gravel bed was built in the flume to give a false floor 0.39 m high. It had an approach slope of 1:20, with a horizontal section about 15 m long before the trench. The trench, perpendicular to the length of the flume, had side slopes of 1:2.75, was 0.32 m deep and 0.45 m wide at its bottom (Figure 1). A smooth stainless steel pipe of 0.042 m nominal diameter machined to 0.0405 m, with wall thickness of 0.00274 m, was placed across the flume in the centre of the trench on a rigid flat metalplate. The downwave end of the false floor ended at a wave absorbing beach made of rubberized animal hair on a slope of 1:8. At a scale of approximately 1:15, the setup was typical of a configuration proposed for production facilities in the Beaufort Sea.

The pipeline model extended across the flume from one wall to the other in the centre of the trench. It was divided into three sections. The sections adjacent to each of the flume walls were fastened securely to the floor of the flume, providing a rigid fixed support between which the 1.0 m long central test section was suspended.

The suspension at each end of the test section consisted of orthogonal pairs of thin, flat beams aligned to the pipe axis and inside the pipe. One pair was compliant perpendicular to the pipe axis in the horizontal plane, the other in the vertical plane. Strain gauges bonded on each pair of beams measured the horizontal and vertical deflections of the suspension at each end of the test section in response to the wave loads imposed on it. The system was calibrated by applying horizontal and vertical forces through a cable and pulley system using known weights.

The suspension system and the strain gauge system turned out to be extremely difficult to make function satisfactorily. The mechanisms holding the pipe sections together via the beams were quite intricate, and appeared to be the source of much of the difficulty encountered. In the end, it was only possible to make functional one beam which measured horizontal force. Even this was not totally satisfactory in that the calibration was different in each direction. The calibrations were, however, repeatable to within ±10%, and tests were conducted with the knowledge that there would be errors of that order due to the instrumentation.

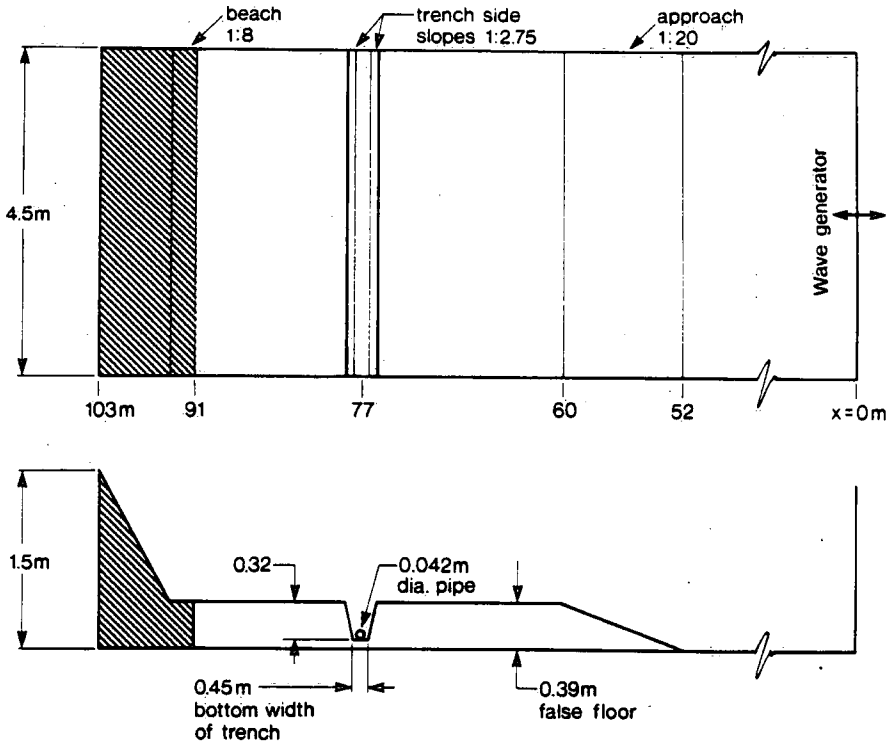

Figure 1. Plan and side views of experimental setup.

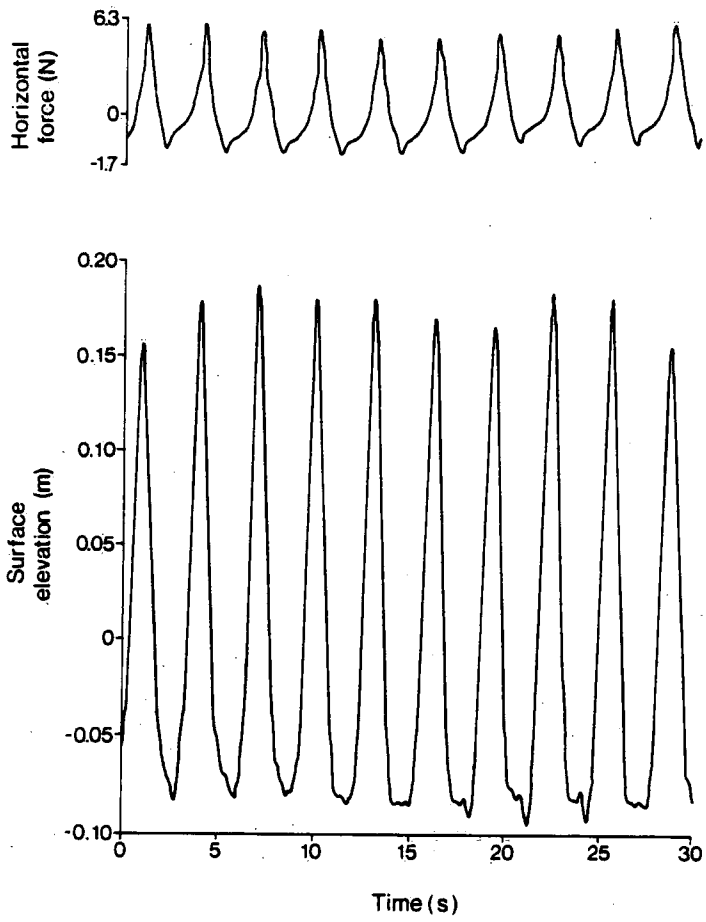

Figure 2. Sample time series of surface elevation and horizontal force, d = 0.61, f = 0.305 Hz, H = 0.223 m.

The pipe was subjected to a series of regular wave conditions up to the proposed prototype design conditions of H = 5.7 m and T = 12 s in a water depth of 30 m, suitably adjusted using a TMA transformation (Hughes and Miller 1987) for depths of 4.5 and 9.0 m. Conditions that produced breaking waves in the flume were eliminated from the analysis.

Each test was started from still water conditions, and the sampling began after about six waves had passed the test section. For each test, 1024 data points were collected at a rate of approximately 20 samples per second from the strain gauges and from the capacitance wave staff positioned directly over the centre of the instrumented section of the pipe. Each time series was examined visually, and any of poor quality (for example, excessive noise) were rejected. A sample time series is shown in Figure 2. The time series were then processed with a computer analysis program to determine the coefficients, using the least squares technique.

In order to gain insight into the overall flow field in the trench, a series of flow visualization tests were conducted in a small glass-walled wave flume (Skafel and Bishop 1989). The geometry of the trench was the same as that used for the force tests, but at a scale of 1:60, relative to the prototype. Velocities were estimated in the trench and on the flat bed using a tracer of fine mica and titanium dioxide particles. The flow was observed visually and recorded on video. Selected cases on the video were examined frame-by-frame to estimate the tracer velocity near the bed.

4.0 RESULTS

From the flow visualization tests it was found that, in spite of the relatively steep side walls, there was no separation at the transition from the flat sea bed. In the vicinity of the pipeline, an eddy formed on the lee side during each half cycle, and was subsequently shed in the next half cycle, similar to that described by Jacobsen et al. 1984. Under all conditions tested the eddy was approximately three or four pipe diameters in size.

The magnitude of the velocity near the bed was estimated from the movement of the tracer material. Although the use of tracer material considerably denser than water resulted in underestimates of the true water particle velocity, it is expected that relative speeds could be determined (Clayton and Massey 1967). The velocity at the bottom of the trench was found to be closely approximated by the bottom velocity at the same total depth, if a simple correction for shoaling was made for the wave height over the trench relative to the measured height over the surrounding flat bottom. This finding suggests that the large body of information available for forces on pipelines on flat beds can be used to estimate loadings due to waves on pipelines in trenches. Furthermore, for the tests reported herein, it is appropriate that the velocity used in the Morison equation be that computed at the depth of the pipe centreline, based on the measured water surface elevation directly over the pipe.

For the force tests, the calculated drag and inertia coefficients along with the corresponding values of K, β and Re are given in Table 1. The values of C_d are plotted as a function of K

TABLE 1. Wave Forces on a Pipeline in a Trench; Summary Data

d* (m)	f (Hz)	H (m)	K	β	Re	C_d	C_m
0.31	0.382	0.100	7.4	580	4300	2.10	3.50
0.31	0.382	0.069	6.8	580	4000	1.80	2.60
0.31	0.477	0.125	8.8	730	6400	1.20	3.10
0.31	0.629	0.118	5.5	960	5300	2.60	3.60
0.31	0.629	0.095	4.5	960	4300	2.30	3.40
0.31	0.305	0.089	9.5	470	4400	2.90	2.60
0.61	0.305	0.119	13.0	470	5800	2.30	1.40
0.61	0.324	0.187	13.0	500	6200	2.20	2.10
0.61	0.305	0.258	25.0	470	12000	2.60	3.30
0.61	0.382	0.088	6.9	580	4000	2.30	1.40
0.61	0.305	0.223	22.0	470	10000	3.00	3.30
0.61	0.382	0.139	11.0	580	6400	2.10	1.50
0.61	0.382	0.223	17.0	580	9800	1.90	2.00
0.61	0.382	0.177	13.0	580	7800	1.90	1.90
0.61	0.477	0.078	4.4	730	3200	1.90	1.50
0.61	0.477	0.145	8.1	730	5900	2.90	1.80
0.61	0.477	0.224	12.0	730	8600	3.60	3.40

* Depth of the false floor, i.e., add 0.32 m for the depth of the trench floor

in Figure 3, and the C_m in Figure 4; both show considerable scatter and no particular trend. Values of C_d vary from 1.2 to 3.8, with a mean of 2.3 and a standard deviation of 0.6. Values of C_m vary from 1.4 to 3.6, with a mean of 2.7 and a standard deviation of 0.8. The range of K covered in the tests was from 4 to 25, and β from 470 to 960.

5.0 DISCUSSION

The drag coefficients found from these tests, while showing scatter, have an average of 2.3 for K in the range of 4 to 25. This is slightly higher than reported by Sarpkaya and Rajabi (1979) for the same range of K. However, their data suggests that the lower β values reported here should produce C_d values higher than theirs which peak at about 2.2 for β = 2840. The results of the present experiments do not show any dependence on β, although such a dependence is clearly

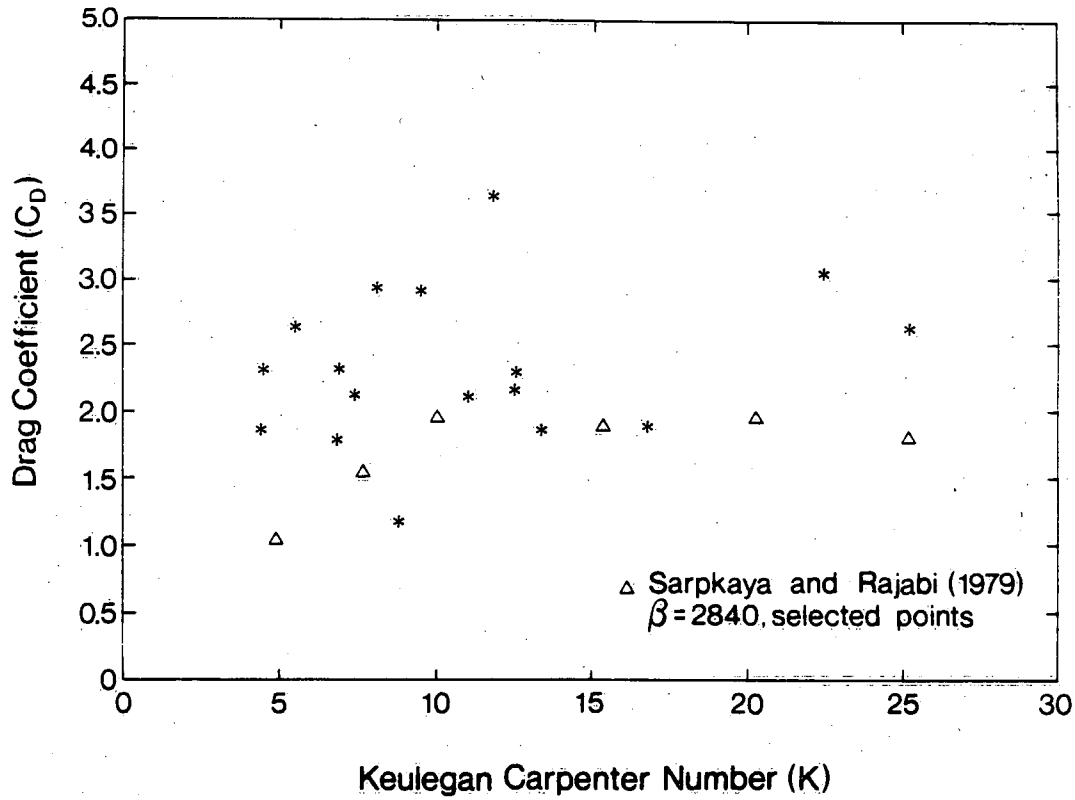

Figure 3. Drag coefficient (C_d) vs Keulegan Carpenter number (K).

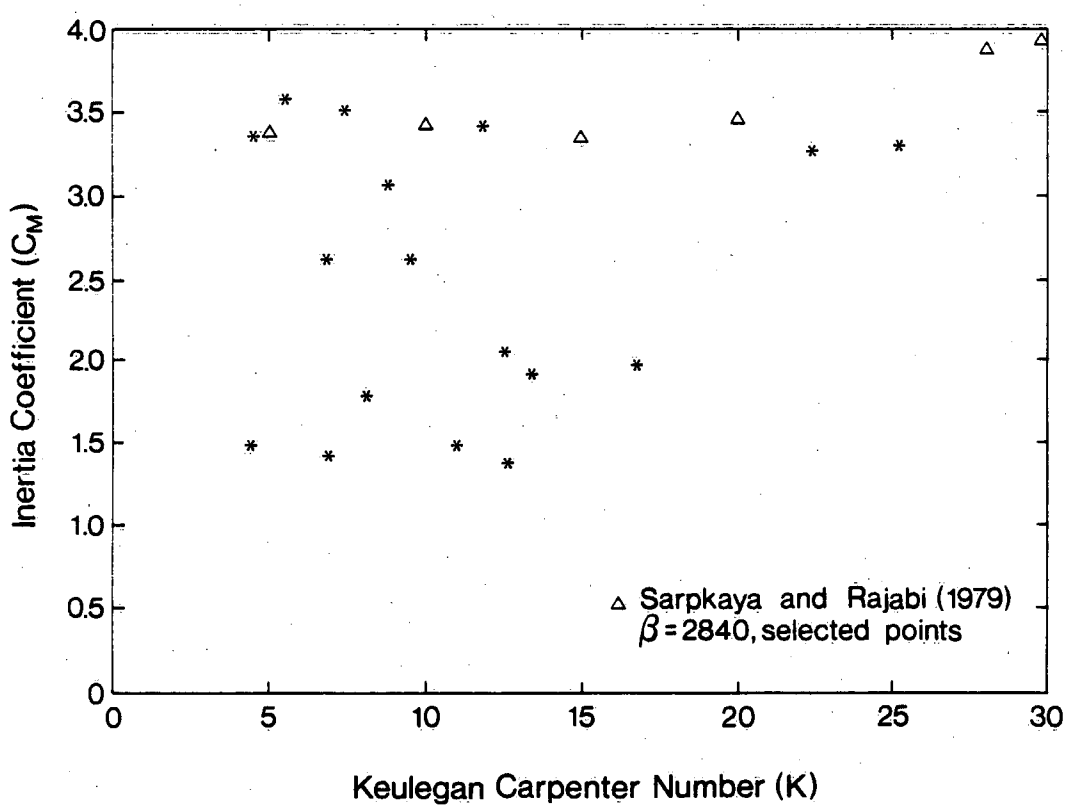

Figure 4. Inertia coefficient (C_m) vs Keulegan Carpenter number (K).

evident in their data. Their results were obtained in very well controlled oscillating water tunnel tests, and show remarkably little scatter. Littlejohns and Spencer (1982) show results from field tests with markedly lower values of C_d that vary from about 0.5 to 1.5. Their data are for prototype conditions, for which they report no dependence on Re (or equivalently, β). The values of Re for their tests are substantially higher than the values in the present experiments. Bryndum et al. (1983) report C_d values in the range of 2 to 2.5 for regular waves (the pipe was oscillated in still water) in the same range of K, but at substantially higher values of Re. All of these tests except the present ones were done on a flat bed.

The inertia coefficients determined from the present tests have an average of 2.7, and also show scatter. Both Sarpkaya and Rajabi and Bryndum et al. report C_m values in the range of 3 to 4, with an increasing trend for values of K beyond the range tested here. Littlejohns and Spencer report C_m values around 2.5, with some scatter. There is no evidence of a dependency on β in the C_m data of Sarpkaya and Rajabi, as there was with the C_d data.

While the present results exhibit considerable scatter, they do fall into the same range of values reported earlier by others, under the assumption that the velocity at the bottom of the trench is the same as that at a flat bottom of the same total depth.

6.0 APPLICATION

The tested trench configuration has been proposed for Beaufort Sea application in water depths greater than or equal to 9 m. Furthermore, similar configurations would be used in water depths from 9 m inshore to landfall. At the bottom of a 4.75 m deep trench in an ambient water depth of 9 m, with wave conditions H = 3.5 m and T = 12 s, values of the dimensionless parameters are K = 26 and β = 19000. Thus the tests here were conducted at slightly lower values of K than in the prototype, and at order of magnitude lower values of β than in the prototype. The results of Sarpkaya and Rajabi suggest the prototype drag coefficient will be somewhat lower than the present results indicate, but the inertia coefficient will be unchanged. The trend for C_d values to decrease with increasing β, shown in their results, and the lower values of C_d reported by Littlejohns and Spencer for prototype conditions suggest that the values obtained in the present tests represent values that would not be exceeded in the corresponding prototype Beaufort Sea conditions.

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