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WAVE FORCES ON A PIPELINE IN A TRENCH

by

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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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ABSTRACT

Wave forces on a model pipeline have been measured. The pipeline was located in a trench, the configuration of which is similar to that proposed for oil and gas production in the Beaufort Sea. It was found that the drag and inertia coefficients were similar to results reported in the literature for flat beds assuming the water depth was that of the total water column including the trench. Flow visualization tests confirmed that the horizontal velocities were about the same as would be found on a flat bed with the same total depth.

Key Words: wave forces, pipelines, trenches

RÉSUMÉ

Les forces exercées par les vagues sur un pipeline modèle ont été mesurées. Le pipeline était déposé dans une tranchée qui avait une configuration semblable à celle proposée pour les gazoducs et les oléoducs proposés pour la mer de Beaufort. Il a été constaté que les coefficients de résistance et d'inertie sont semblables à ceux rapportés dans le cas de pipelines posés directement sur le fond, dans l'hypothèse que la profondeur d'eau est égale à la colonne d'eau totale, y compris la tranchée. Les tests de visualisation de l'écoulement ont permis de confirmer que la vitesse horizontale est sensiblement la même qu'il y ait tranchée ou non, à une même profondeur totale.

Mots clés : force des vagues, pipelines, tranchées

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. To avoid this problem it has been proposed to place the pipelines in trenches below the maximum anticipated depth of scour from ice floes. Therefore, 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 been focused on the problem of 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 the trench depth is of approximately the same size as the pipe diameter, a totally 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.

A 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 as well as possible 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) is particularly suitable for analysis by computer, and was used in the present study. In this method the time series of the force is fitted to the time series of velocity and acceleration by the method of least squares. The Morison equation is written in the form

$$[3] \qquad F_i(t) = (\ \alpha_1 A_i + \alpha_2 U_i |U_i|\ )$$

where the α's include the drag and inertia coefficients (compare with equation 1). The subscripts denote the ith data point in the time series. Minimizing the quantity

$$\sum_i \{ F_i - \alpha_1 A_i - \alpha_2 U_i |U_i| \}^2$$

leads to the definition of three matrices:

$$\gamma = \left(\sum_i F_i A_i \qquad \sum_i F U_i |U_i| \right)$$

$$M = \begin{pmatrix} \sum_i A_i^2 & \sum_i A_i U_i |U_i| \\ \sum_i A_i U_i |U_i| & \sum_i U_i^4 \end{pmatrix}$$

$$\alpha = (\alpha_1 \quad \alpha_2)$$

and the matrix equation:

$$\gamma = \alpha M$$

The solution for α is:

$$\alpha = \gamma M^{-1},$$

and hence the drag and inertia coefficients can be found. The lift coefficient is found in a similar fashion.

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 metal plate. 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 large stiffness of the pipe test section compared to the stiffness of the instrumented suspension beams ensured that wave loads on the test section caused lateral movement of the test section through strain of the suspension beams rather than bending of the test section.

The dynamical response of the pipeline was investigated to ensure that its characteristics did not obscure the measurements of drag and inertia coefficients. Two types of response were examined, the case of a rigid pipe mounted on springs (the compliant beams), and a flexible pipe with several possible boundary conditons.

Although both types of response were checked, the former was considered to be a more realistic representation of the system. In this description of the system each beam was cantilevered at the end attached to the outer pipe section and guided at the test section end. The spring constant was found from the beam geometry and material properties. The computed natural frequency was then adjusted for immersion in water (Sarpkaya and Isaacson 1981). This value was used in computing the dimensionless "reduced velocity",

$$V_r = U/f_n D,$$

where f_n is the natural frequency in Hertz. Various researchers have found that the reduced velocity must exceed values of about 4.0 for vibration to be important (see, for example, Raven et al. 1985). The largest value during our experiments using the rigid pipe description was about 0.24. Similar computations were made for the flexible pipe approximation. The simply supported case gave the lowest mode of vibration, so it was used, and the reduced velocity was estimated to be about 0.1. Based on these calculations, it was assumed that pipe vibrations were not important during our tests. By comparison, Sumer et al. (1989) kept the reduced velocity in the range from 3 to 8 to investigate the vibration response of a pipe over a scour trench.

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%, so that a series of tests were conducted with the knowledge that there would be errors of that order due to the instrumentation.

The pipe was subjected to a series of regular wave conditions up to 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.

3.1 Flow Visualization

In order to gain insight into the overall flow field in the trench, a series of flow visualization tests were done in a small glass-walled wave flume (Skafel and Bishop 1989). The geometry of the trench was the same as used for the force tests, scaled down by 1:4. Velocities were estimated in the trench and on the flat bed using a tracer of fine mica and titanium dioxide particles. There were two findings from these tests that are important to the force tests. Firstly, there was no flow separation at the top corners of the trench, so there was no concern that the pipe would be in a separation zone. Secondly, the velocity at the location of the pipe in the trench was the same as the velocity adjacent to a flat bottom, at the same total depth. (The average measured velocity under the wave crests adjacent to the trench floor was 0.93 m/s, and adjacent to the flat bed was 1.08 m/s, in prototype units. The overall accuracy of the measurements was estimated to be 15%.) Based on these results, the computed velocity to use in the Morison equations is that assuming a flat bottom at the total depth of the ambient water plus the trench.

4.0 RESULTS

The 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 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 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. Much of the earlier work shows considerable scatter and will not be compared here (see Skafel, 1985, for some examples).

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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SYMBOLS

A	horizontal water acceleration at the pipe location
C_d	drag coefficient
C_m	inertia coefficient
C_v	lift coefficient
d	water depth above false floor
D	pipe diameter
f	wave frequency (1/T)
f_n	natural frequency of the pipe
F_h	horizontal force on pipe
F_v	vertical force on pipe
H	wave height
k	surface roughness of pipe
K	Keulegan Carpenter number (UT/D)
Re	Reynolds number (UD/ν)
t	time
T	wave period
U	horizontal water velocity at the pipe location
V_r	reduced velocity (U/f_nD)
β	frequency parameter ($D^2/T\nu$)
ν	kinematic viscosity
ρ	water density

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

FIGURES

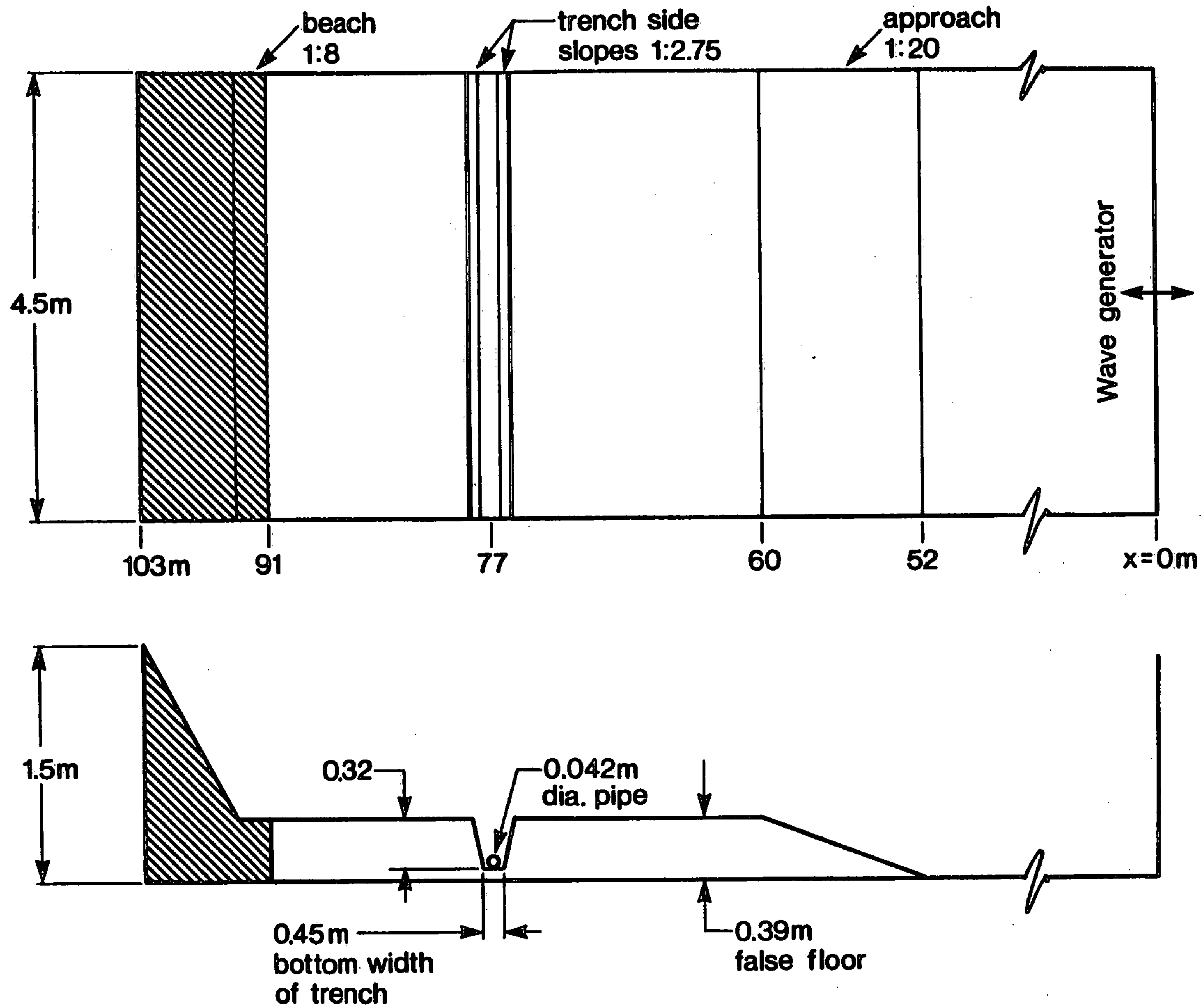

beach
1:8
trench side
slopes 1:2.75
approach
1:20
Wave generator
4.5m
103m
91
77
60
52
x=0m
1.5m
0.32
0.042m
dia. pipe
0.45m
bottom width
of trench
0.39m
false floor

Fig. 1.

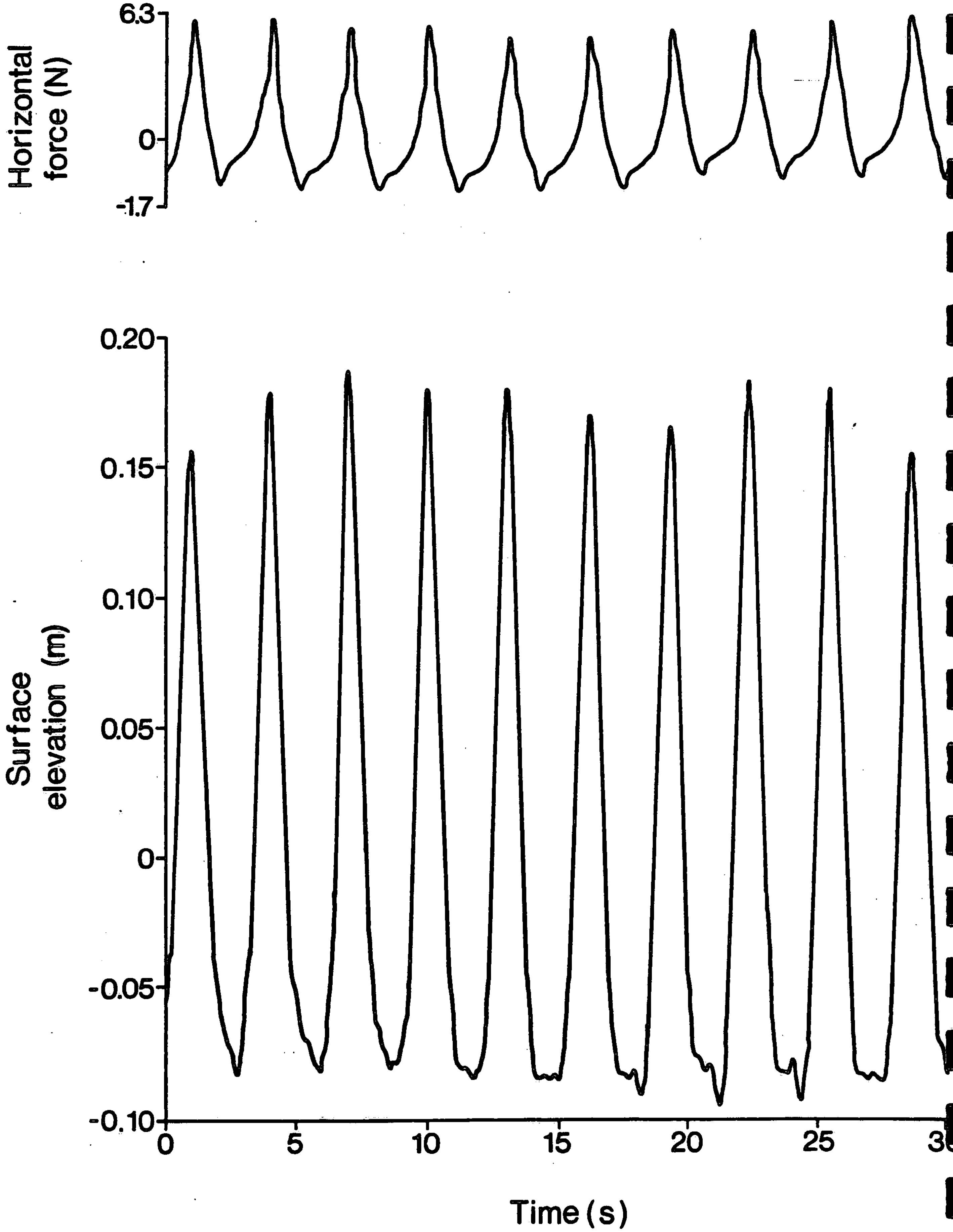
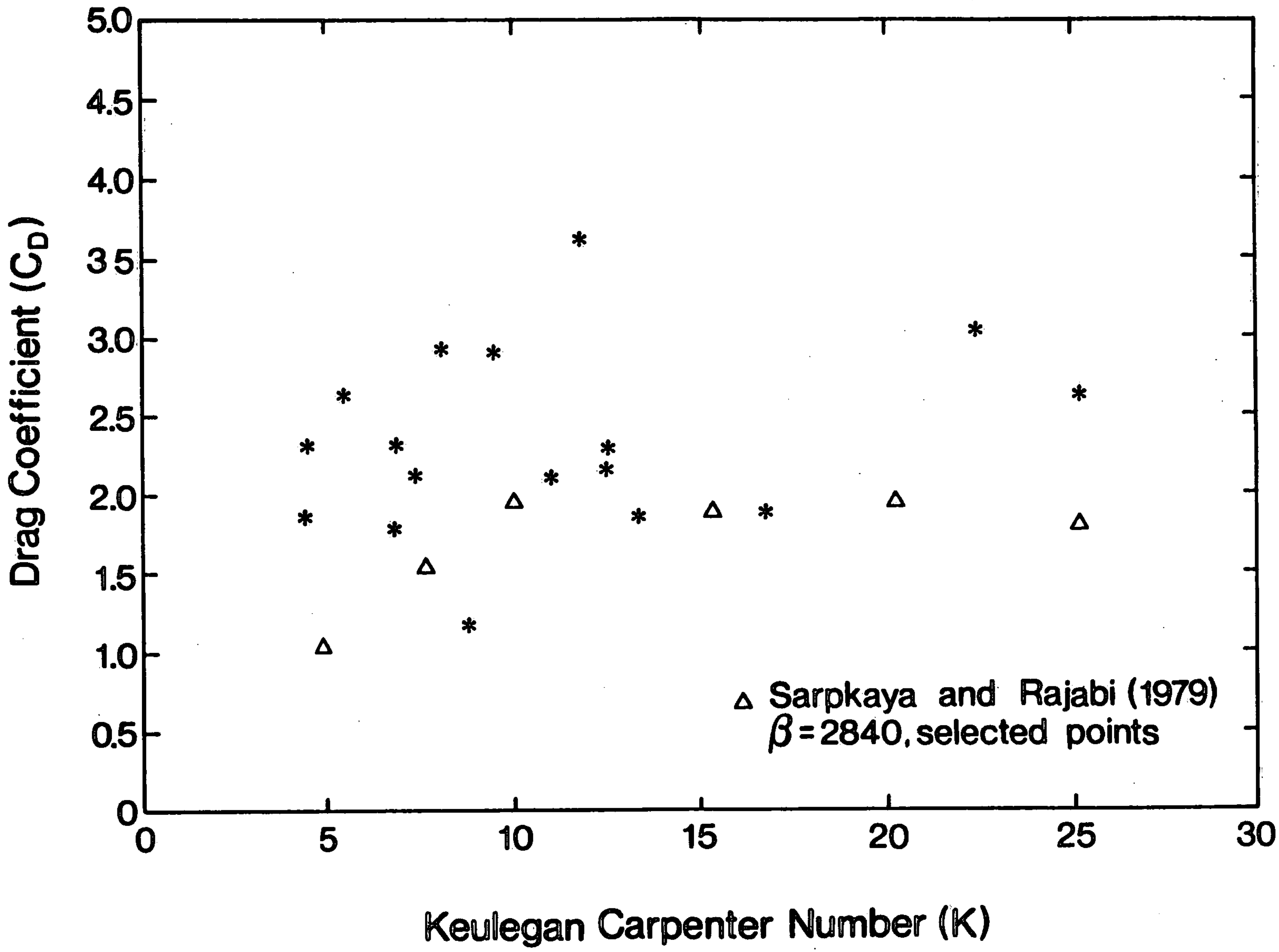

6.3
0
-1.7
Horizontal force (N)
0.20
0.15
0.10
0.05
0
-0.05
-0.10
Surface elevation (m)
0
5
10
15
20
25
30
Time (s)

Fig. 2



5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
Drag Coefficient (C_D)
0
5
10
15
20
25
30
Keulegan Carpenter Number (K)
△ Sarpkaya and Rajabi (1979)
β = 2840, selected points

Fig. 3

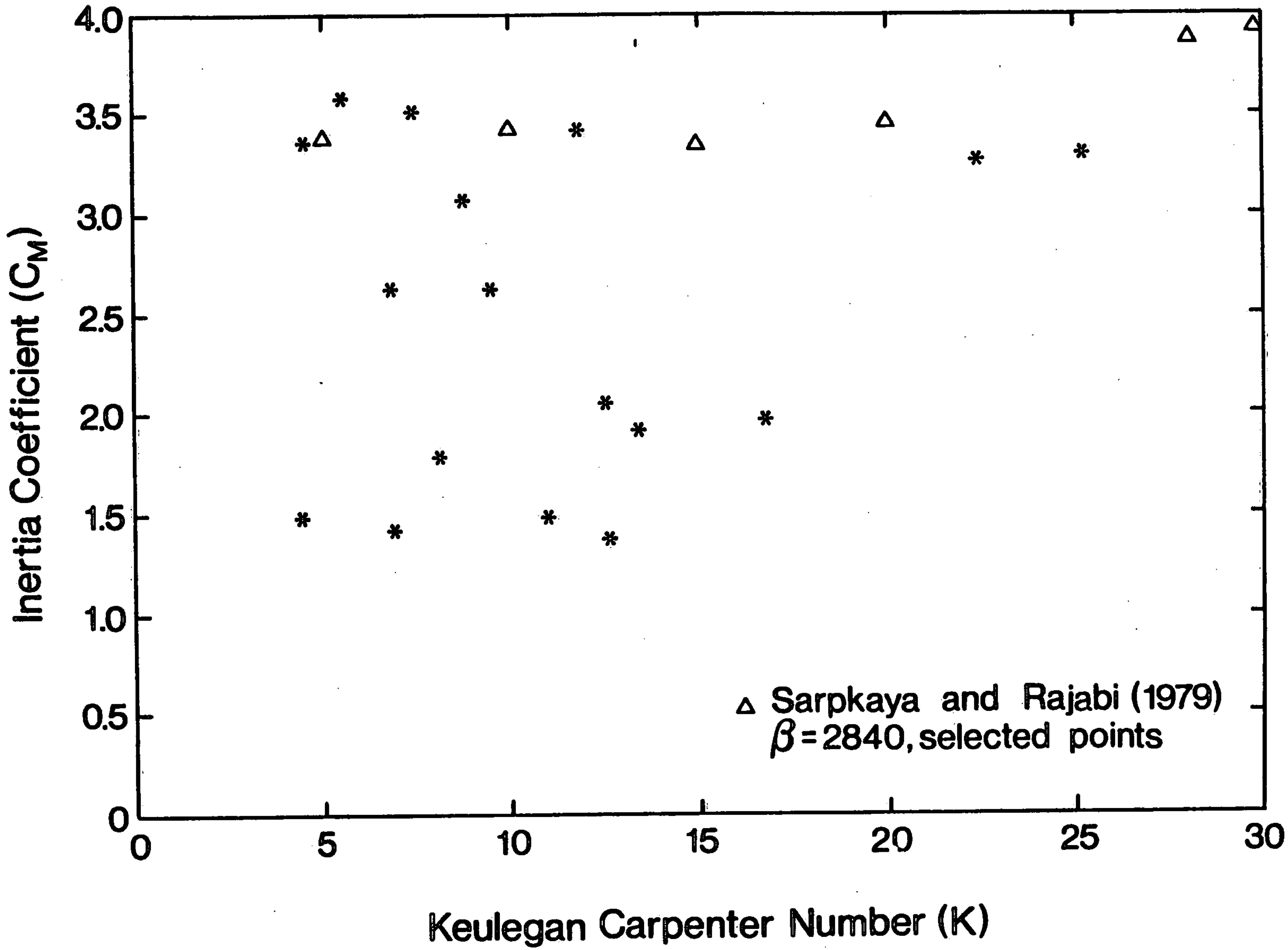

4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
Inertia Coefficient (C_M)
0
5
10
15
20
25
30
Keulegan Carpenter Number (K)
△ Sarpkaya and Rajabi (1979)
β=2840, selected points

Fig. 4