

CHAPTER 159

WAVE FORCE AND MOVEMENT CALCULATIONS FOR A FLEXIBLE OCEAN OUTFALL PIPELINE

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ABSTRACT

The design process for the calculation of wave forces and movements for a flexible (plastic) ocean outfall is described. The design procedure is illustrated using a case study of the design of two High Density Polyethylene (HDPE) pipelines of 0,9 m and 1,0 m OD (4 290 m and 5 450 m long) constructed at Richards Bay, South Africa, to dispose of dense and buoyant effluent respectively.

The pipeline anchor weights are based on the 1 in 1 year wave forces on the pipeline, implying that the pipeline is allowed to move during its design life. Special star anchor weights are used which keep the pipe clear of the bed while maintaining the stability of the pipeline.

Friction tests were undertaken with a section of pipeline and two star weights, above water on concrete and sand and below water on sand, to determine realistic friction coefficients for the pipeline design. The results of these tests are summarised in this paper. It was found that the mean friction coefficient for submerged star weights on sand was 0,75.

The movements of sections of the 0,9 m OD pipeline were calculated using a finite difference computer programme developed by Prof I Larsen and the results are summarised in the paper. It was found that movements of 1 to 2 m could occur under design wave conditions (50 to 100 year waves) and these were considered acceptable provided that the pipeline was not obstructed by rock outcrops.

1. INTRODUCTION

Two High Density Polyethylene (HDPE) pipelines of 0,9 m and 1,0 m OD have been constructed at Richards Bay, South Africa, (160 km North of Durban) to dispose of dense and buoyant effluent respectively. The dense and buoyant effluent pipelines are 4 290 m and 5 450 m long (measured from the pump station) and discharge at depths of 24 m and 29 m respectively. The dense effluent consists primarily of waste gypsum from a fertiliser plant while the waste from a large paper pulp mill accounts for the bulk of the buoyant effluent.

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During the initial design phase NRIO was involved with the site investigations and the effluent dilution calculations. When the contract went out to tender, flexible HDPE pipelines were proposed and subsequently constructed. These pipelines were originally designed using the Scandinavian Design Procedure (SDP) for flexible pipelines which was based on a large amount of experience with flexible pipelines in the Scandinavian countries. Since previous pipeline experience in South Africa was with rigid concrete or steel pipelines, NRIO reviewed the flexible pipeline design procedure. NRIO was then commissioned to check the "as built" design of the Richards Bay outfall marine pipelines. To do so, the design wave conditions, wave forces and consequently the required weighting along the pipelines were calculated. Friction tests were carried out to determine realistic friction coefficients and the expected movements of various sections of the pipelines were determined.

2. WAVE FORCE CALCULATIONS

2.1 The Scandinavian Design Procedure (SDP)

The forces on the pipelines and the weights of the anchor blocks were calculated in accordance with the SDP. Firstly, it should be noted that there is no "design code" for flexible pipelines. Present design is based on experience gained in Scandinavia over the past 20 years (Janson, 1974, 1978; Janson and Larsen, 1979; Björkland, 1983).

The "design procedure" should include the following steps:

- i) The choice of HDPE or PP (Polypropylene) material type is dependent on the effluent temperature.
- ii) Pipe wall thickness is determined to ensure that stresses due to internal pressure do not exceed a given value which depends on material, temperature and service life as well as adequate safety against buckling due to external and internal loads.
- iii) **Assuming** a friction factor $f = 1$ stability against sliding requires:

$$f(w - F_L) \geq F_H \quad (1)$$

where: w = submerged anchor weights

F_L = lift force

F_H = horizontal force (maximum combined).

Thus for $F_L = 0$ (pipe well clear of the bed) it follows:

$$w \geq F_H$$

or the anchor weight is equal to (or greater than) the horizontal force. If $f = 1$ and square or star-shaped anchor blocks are used the safety against overturning of loose anchor blocks is the same as the safety against sliding.

- iv) It is accepted that the pipe may move once or twice a year. This means that, say, the once-a-year occurring **maximum** wave heights must be used to determine the forces on the pipe, that is,

$H_{\max} \approx 2 H_{\text{mo}}$, where H_{mo} is the characteristic wave height based on six-hourly records with an occurrence of once a year.

- v) In determining the wave forces it is assumed that the pipe will always be clear of the bottom (resting on the anchor blocks which protrude usually more than $D/4$ from the pipe, where D is the pipe diameter). If the anchor blocks were to sink into the sea bottom because of local scour, a lift force would develop which would increase at smaller pipe clearances. If the lift force exceeded the anchor weight, the pipe would become buoyant and would lift off the bottom until the vertical forces found a new equilibrium, re-establishing a clearance. Deposition of sand beneath the pipe is unlikely because of increased water velocity and extra turbulence in the area. If it did happen the same lifting process would occur. If the sea bottom were to be raised in the area by general accretion, the pipe would be buried and the wave forces reduced accordingly.
- vi) Pipe force calculations may be based on the simplified formulae and refraction graphs contained in Janson (1974) if more detailed information is not available. However, more reliable results can be obtained by using a higher-order wave theory in the shallower water and local wave conditions or wave conditions converted by actual refraction diagrams to the site, taking into account depth-limiting conditions. This information is then fed into the basic Morison equation.

For the SDP the horizontal (parallel to the bed) wave forces on a pipeline are calculated using the Morison equation. Based on the assumption of minimum ($D/4$) clearance before horizontal motion can take place, the following force coefficients are used:

$C_M = 2$ (inertial/added mass coefficient)

$C_D = 0,7$ ($C_D = 0,33$ is the original SDP value which has been in use for almost 20 years. In view of the new rules of Norske Veritas for oil and gas pipelines, which may be somewhat conservative, and the tests of Sarpkaya, a value of 0,7 was used in these studies; Larsen, 1984)

$C_L = 0$ (lift coefficient).

- vii) Knowing the anchor blocks and the pipe characteristics, it must be ensured that the spacing between the blocks will be such that the permissible bending will not be exceeded (Europlast, 1984). It should then be determined whether, for the design life of the pipe, for example 50 years, the total movement and resulting maximum bending moments and strains will be acceptable, that is, movement should preferably not exceed a few metres, short-term strain should not exceed 1,5 per cent and long-term deformation after 50 years must not exceed 6 per cent (Janson and Larsen, 1979 and Europlast, 1984). Recent data (private communication Prof Larsen) indicates that short-term strains $\leq 2,5$ per cent would be acceptable. Further details of the SDP are given by Pos (1986).

For the Richards Bay pipelines star-shaped anchor weights were used. It was hypothesised that these weights would provide greater

resistance against sliding than the more conventional square anchor weights. Figure 1 shows typical star weights as installed on a section of the Richards Bay pipelines.



Figure 1: Typical star weights, Richards Bay HDPE pipeline

2.2 Calculation of Anchor Block Weights

The SDP states that the weighting required at a particular location along a pipeline is equivalent to the maximum horizontal 1:1 year wave force expected to occur at that position. The maximum horizontal wave forces along the 0,9 m and 1,0 m marine pipelines, for 1:1 year design waves were calculated using the programme "PIPE" installed on the CSIR's CDC computer. The above programme uses the Vocoidal wave theory (Swart and Laubser, 1978) to calculate the horizontal velocities and accelerations and the Morison equation to calculate the maximum combined horizontal wave forces on the marine pipelines.

The 1:1 year maximum horizontal wave loadings along the 0,9 m OD Richards Bay outfall marine pipeline are given in Table 1 as a sample of the results*. For each chainage position the following data are given in Table 1:

- (a) The water depth, wave height, angle of incidence, maximum crest bottom velocity, maximum trough bottom velocity, maximum absolute bottom acceleration (12 s period waves were used throughout).
- (b) The maximum drag and inertial and combined drag and inertial (incorporating phase effects) horizontal force (kN/m).
- (c) The above horizontal force expressed as a percentage of the weight of water (per m length) displaced by the pipe.

*Details regarding the design wave conditions used for the force calculations are given in CSIR (1985).

(d) The installed weighting.

Comparison of the last two columns in Table 1 shows that except for the first section of the pipeline, the installed weights are larger than those calculated using the SDP, which requires that the submerged anchor weight be equal to the maximum horizontal force for the 1:1 year wave condition (if it is assumed that for a pipe mounted D/4 above the bed inertial, drag and lift coefficients of 2, 0,7 and 0 respectively apply and a friction coefficient of 1,0 was used for these calculations). In a subsequent study Pos (1986) has suggested that based on the work of Sarpkaya (1977) and DNV (1981) inertial, drag and lift coefficients of 2, 0,8 and 0,4 would be more appropriate. The lift force (in phase with the drag force) particularly will have an effect on the pipeline stability in those regions where the drag and inertial forces are of similar magnitude, such as the diffuser sections.

Table 1: 1:1 Year maximum horizontal wave loading along the 0,9 m OD pipeline

Chainages along the pipe	Water depth (m)	Wave height (m)	Angle of incidence (degrees)	Maximum crest bottom velocity (m/s)	Maximum through bottom velocity (m/s)	Maximum absolute bottom accel. (m/s ²)	Maximum inertial force (kN/m)	Maximum drag force (kN/m)	Max. combined drag and inertial horizontal force (kN/m)	Horizontal force expressed as a percentage of displ. weight	Installed weights expressed as a percentage of displ. weight
Ch 1800	8,5	6,6	48	3,29	-1,23	3,98	3,05	1,93	4,03	63,0	50,0
Ch 1250	12,0	8,9	50	3,61	-1,68	3,83	3,82	2,47	3,99	62,4	50,1
Ch 1500	13,8	8,6	51	3,19	-1,72	2,97	3,01	1,99	3,11	48,7	50,1
Ch 2000	16,7	8,6	52	2,78	-1,86	2,29	2,35	1,55	2,38	37,3	40,1
Ch 2500	18,0	8,6	52	2,64	-1,93	2,08	2,13	1,40	2,15	33,6	40,1
Ch 3000	19,0	8,6	49	2,54	-1,98	1,94	1,91	1,18	1,91	29,9	40,1
Ch 3500	20,0	8,6	49	2,44	-2,04	1,81	1,78	1,10	1,79	27,9	30,8
Ch 4000	21,0	9,3	52	2,53	-2,22	1,89	1,94	1,28	1,95	30,4	30,8
Ch 4290	24,0	10,0	53	2,41	-2,38	1,79	1,86	1,20	1,92	30,0	33,2
Ch 4290 (0,61 m OD)	24,0	10,0	53	2,41	-2,38	1,79	0,91	0,84	1,01	32,1	33,2

Displaced mass of 0,9 m OD pipe = 652,1 kg/m

Displaced mass of 0,61 m OD pipe = 299,6 kg/m

The pipe is assumed to be D/4 m clear of the bed

$C_m = 2,0$

$C_D = 0,7$

$C_L = 0,0$

3. FRICTION TESTS

3.1 Introduction

In order to check the SDP assumption that the friction factor $f = 1$, it was decided to carry out tests, using a section of the pipeline with associated star weights, to determine realistic friction coefficients for a range of test conditions.

These tests were carried out at Richards Bay on 23 and 24 April 1985 to establish the friction factors associated with the sliding of the concrete star anchor weights, **out of water** over concrete and sand, and **under water** over sand. The above-water friction tests simulated under-water conditions and enabled a range of surfaces from smooth concrete to dry sand to be monitored visually and photographically. Full details regarding the test configurations, procedures and results are given in CSIR (1985), while the main findings are summarised below.

3.2 Test Configurations

The test configuration used for the above-water concrete and sand friction tests is shown in Figure 2. The test rig consisted of a 3 m section of 900 mm OD (40 mm wall thickness) High Density Polyethylene (HDPE) pipe to which was attached two half star weights spaced 1 m apart symmetrical about the centre of the length of pipe. The two half star weights were clamped to the pipe using two steel clamps. The reason for using the half star weight test rig for all the above-water friction tests is that its weight above water is approximately equal to the submerged weight of the whole star weight rig. It was thus proposed that this test arrangement could be used to simulate, above water, the submerged sliding behaviour of a section of the pipeline.

The test configuration used for the underwater friction tests was the same as that used for the above-water tests except that now full star weights were used. For both configurations the pulling force was applied at the level of the pipe axis.



Figure 2: Above-water friction test configuration

3.3 Test Procedures

The above-water friction test procedures both for the concrete and sand friction tests were virtually identical. The ends of the half loop of chain was shackled to the steel clamps of the test configuration as shown in Figure 2. This was then connected to the pulling cable of a mobile crane via a load shackle. A continuous load (in the form of milli-volts from the strain gauges) versus time plot was obtained for each test via a pen recorder interfaced with the load shackle amplifier. The milli-volt versus time plots were converted into load (in kN) versus time plots using a predetermined calibration curve.

To determine the displacement of the star weights with time two survey staffs were placed on the ground, parallel to the pulling cable, with the beginning of each staff adjacent to the toe of a star weight. For

the tests on concrete the staffs were aligned with the front toes (closest to the crane) of the weights (see Figure 2), while for the above-water sand tests the staffs were aligned with the back toes of the weights. A stop-watch was started at the beginning of the test and the time noted at 0,1 m displacement increments as each weight displaced relative to its staff.

For the tests on concrete, consecutive tests were merely started with the test rig in the position corresponding to the end of the previous test. For the tests on sand, however, after each test, the test rig was lifted and positioned on a section of undisturbed sand before the next test was started.

The underwater friction tests were performed in a large flooded pit with a sand bed using the test configuration described previously. The crane cable was connected to the pulling cable of the test configuration via the load shackle. As for the above-water tests, a continuous load versus time plot was obtained for each test via the load shackle and its peripherals. After each test the test rig was lifted and repositioned in an undisturbed section of the basin.

To determine the displacement of the test rig with time a survey staff was placed on the ground, parallel to the pulling cable, with the beginning of the staff adjacent to a chalk mark on the load shackle. A stop-watch was started at the beginning of the test and the time noted at 0,1 m displacement increments, as the load shackle displaced relative to the staff. A note was also made of the time at which the star weights had stopped displacing laterally and were only tilting.

3.4 Friction Coefficient Calculation Procedure

The friction coefficients were calculated using Equation 1. Setting the lift force F_L to zero and assuming that the pipe is on the point of motion, Equation 1 reduces to:

$$Wf = F_H \quad (2)$$

and thus

$$f = \frac{F_H}{W} \quad (3)$$

where, in this case,

F_H = horizontal pulling force

W = weight of test rig

f = friction coefficient

3.5 Above Water Friction Test Results

The friction test results for smooth (surface roughness + 1 mm) and rough (surface roughness + 5 mm) concrete are summarised in Tables 2 and 3 respectively. The movement of the anchor weights across the concrete consisted of a number of individual sliding events. Each event consisted of a load build-up phase during which the star weights

tilted over slightly and a sliding phase in which the load was released. The mean peak pulling force (associated with the initiation of sliding) of individual sliding events and the corresponding friction coefficients (calculated by means of Equation 3) for these tests are summarized in Tables 2 and 3. The test mean friction coefficients are 0,82 and 0,76 for the smooth and the rough concrete tests respectively.

Table 2: Friction Coefficients for Smooth Concrete

Test No.	Mean peak pulling force of individual sliding events (kN)	Mean friction coefficient
2-1	20,2	0,83
2-2	20,6	0,84
2-3	19,1	0,78
Test mean	20,0	0,82

Table 3: Friction Coefficients for Rough Concrete

Test No.	Mean peak pulling force of individual sliding events (kN)	Mean friction coefficient
3-1	17,9	0,73
3-2	18,8	0,77
3-3	18,7	0,76
Test mean	18,5	0,76

The above-water wet and dry sand friction test results are summarised in Tables 4 and 5 respectively. In each table the mean friction coefficients for initial movement (F_{init}), for the displacement range 0 - 0,2 m ($f_{0-0,2}$), 0,2 - 0,4 m ($f_{0,2-0,4}$), 0,4 - 0,6 m ($f_{0,4-0,6}$), >0,6 m, ($f_{>0,6}$) and for the pure tilting phase (f_{tilt}) are given. It is evident that there are three distinct phases during the movement of the anchor weights across the sand, namely:

- (i) an initial pure sliding phase;
- (ii) a sliding and tilting phase in which the weights progressively slide less and tilt more;
- (iii) a pure tilting phase in which the weights tilt over with little or no further forward displacement.

Table 4: Friction Coefficients for Wet Sand

Test No.	f_{init}	$f_{0-0,2}$	$f_{0,2-0,4}$	$f_{0,4-0,6}$	$f_{>0,6}$	f_{tilt}
4-3	0,24	0,31	0,58	0,71	-	0,84
4-4	0,30	0,45	0,66	0,73	-	0,85
4-5	0,34	0,50	0,70	0,82	-	0,93
Test mean	0,29	0,42	0,65	0,75	-	0,87

Table 5: Friction Coefficients for Dry Sand

Test No.	f_{init}	$f_{0-0,2}$	$f_{0,2-0,4}$	$f_{0,4-0,6}$	$f_{>0,6}$	f_{tilt}
5-1	0,13	0,32	0,59	0,75	0,81	0,86
5-2	0,13	0,34	0,60	0,74	-	0,91
5-3	0,11	0,41	0,68	0,84	0,89	0,94
Test mean	0,12	0,36	0,62	0,78	0,85	0,90

3.6 Underwater friction test results

The underwater friction test results are summarised in Table 6. As for the above water tests on sand, the initial movement and the mean friction coefficients for the previously described displacement ranges are given in this table. A sample force and displacement plot is shown in Figure 3. From this plot it is evident that, as for the above water friction tests on sand, the displacement of the submerged anchor weights over the sand bed again incorporate the three movement phases of pure sliding, combined sliding and tilting and pure tilting.

Table 6: Friction Coefficients for Underwater Tests

Test No.	f_{init}	$f_{0-0,2}$	$f_{0,2-0,4}$	$f_{0,4-0,6}$	$f_{>0,6}$	f_{tilt}
6-1	0,45	0,56	0,72	0,86	0,94	0,87
6-2	0,50	0,51	0,65	0,71	0,76	0,70
6-3	0,37	0,51	0,69	0,79	-	0,80
Test mean	0,44	0,53	0,69	0,79	0,85	0,79

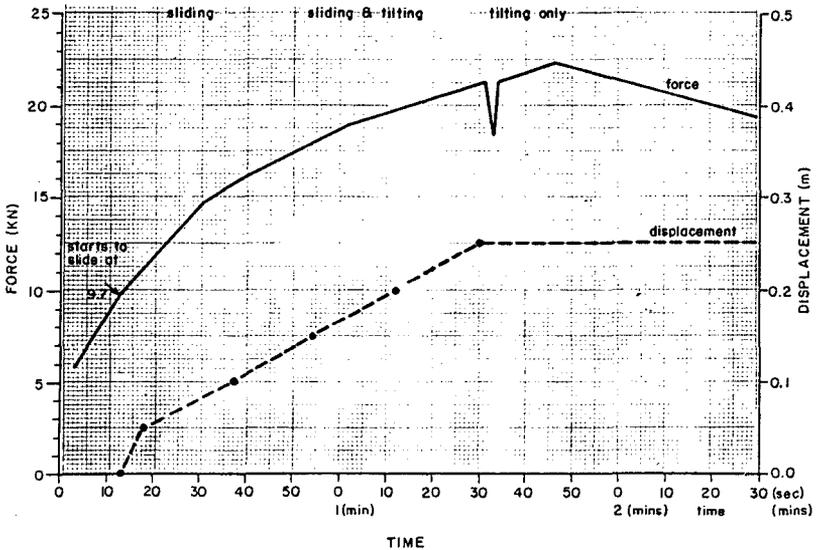


Figure 3: Force and displacement-versus-time plot for underwater friction test No 6-3

3.7 Conclusions

For the smooth and rough concrete tests the mean friction coefficients were 0,82 and 0,76 respectively. The rough concrete friction coefficient of 0,76 would seem the most appropriate for the case for example, of star weights resting on a flat rock reef. This coefficient is significantly less than the SDP assumed friction coefficient of 1. The movement of the anchor weights over such a rock surface will probably consist of a series of sliding events.

The wet sand tests, the dry sand tests and the underwater tests all showed that the movement of the star weights across sand consisted of an initial pure sliding phase, followed by a combined sliding and tilting phase and ended with a pure tilting phase.

Since the pure sliding phase was associated with the digging in of the legs of the star weights, it is thought that the most realistic values to adopt are the $f_{0.2-0.4}$ and $f_{0.4-0.6}$ values. The $f_{>0.6}$ and f_{tilt} values are not thought to be realistic as, due to the high torsional resistance of the pipeline, it is likely that the weights will rotate on the pipe rather than tilt (through any appreciable angle) monolithically with the pipe.

For the above-water tests on sand the wet sand $f_{0.2-0.4}$ and $f_{0.4-0.6}$ values of 0,65 and 0,75 (see Table 4) are thought to be the most representative. For the underwater tests the $f_{0.2-0.4}$ and $f_{0.4-0.6}$ values of 0,69 and 0,79 (Table 6) are thought to be the most appropriate for design purposes.

Based on these data it is suggested that for a sandy sea bottom a value of 0,75 be used, which is significantly lower than the value of 1,0 used when designing the pipeline according to the SDP. It is interesting to note that Lambrekos (1985) also obtained an average maximum friction coefficient of 0,75 for the lateral sliding of a 0,61 m OD steel pipe (with no anchor weights) on a sandy sea bed.

4. PIPELINE MOVEMENT CALCULATIONS

Because the friction tests had shown that friction coefficients could be less than 1, the expected (design wave) movements of the pipelines were calculated for the most critical sections, using a pipeline movement program developed by Prof Larsen (Abbott, Larsen and Verwey, 1977) and modified to incorporate the Vocoidal wave theory (Swart and Laubser, 1978).

For each wave-loading condition investigated a 380 m section of the 0,9 m OD pipeline was modelled using the program. The pipe section was modelled using 39 nodes, that is, with a 10 m spacing between nodes. The time step used throughout was 0,2 s. The relevant design wave data and the results are summarized in Table 7. For each chainage position listed the following data are given:

1. The return period of the design wave conditions used;
2. The MSL water depth, the maximum wave height and the mean angle of incidence of the waves relative to the pipeline axis (12 s period waves were used for all the conditions tested).
3. The pipe OD; for the transition between the pipe and the diffuser runs were done using both 0,9 m and 0,61 m OD pipes;
4. The friction coefficient used (0,5; 0,75 or 1);
5. The installed anchor weighting per metre, expressed as a percentage of the displaced weight;
6. The maximum transient excursion and the maximum residual displacement for one wave cycle (measured in the central 140 m portion of the pipeline). As an example the displacement plot for chainage 4290 for a 1:50 year wave and a relatively low friction factor of 0,5 is shown in Figure 4.

For a friction coefficient of 1 the results showed expected lateral movements of 0,07 to 0,19 m for the 1:1; 0,26 to 0,55 m for the 1:10; 0,65 to 1,09 m for the 1:50 and 0,87 to 1,34 m for the 1:100 year design wave. For a more realistic friction coefficient of 0,75, these movements were approximately double. The total expected movements for design storms can be obtained by accumulating the movements for individual wave heights.

Table 7: Lateral displacements along sections of the 0,9 m OD pipeline

Chainage	Return period (years)	MSL depth (m)	Maximum wave height (m)	Mean angle of incidence (degrees)	Pipe OD (m)	Friction coefficient	Installed weighting expressed as a % of displaced weight	Absolute Displacement	
								Maximum excursion (m)	Maximum residual displacement (m)
1 000	1:1	8,5	6,6	48	0,90	1,0	50,0	0,12	0,05
1 250	1:1	12,0	8,9	50	0,90	1,0	50,1	0,19	0,11
1 250	1:10	12,0	9,5	50	0,90	1,0	50,1	0,30	0,18
2 000	1:100	16,7	12,5	52	0,90	1,0	40,1	1,02	0,51
3 500	1:1	20,0	8,6	49	0,90	1,0	30,8	0,07	0,01
3 500	1:10	20,0	10,6	49	0,90	1,0	30,8	0,26	0,05
3 500	1:50	20,0	11,9	49	0,90	1,0	30,8	0,65	0,18
3 500	1:100	20,0	12,5	49	0,90	1,0	30,8	0,87	0,26
4 000	1:1	21,0	9,3	52	0,90	1,0	30,8	0,09	0,02
4 000	1:10	21,0	11,5	52	0,90	1,0	30,8	0,55	0,09
4 000	1:50	21,0	12,9	52	0,90	1,0	30,8	1,09	0,18
4 000	1:50	21,0	12,9	52	0,90	0,75	30,8	1,85	0,20
4 000	1:100	21,0	13,5	52	0,90	1,0	30,8	1,34	0,17
4 290	1:1	24,0	10,0	53	0,61	1,0	33,2	0,07	0,06
4 290	1:10	24,0	12,3	53	0,61	1,0	33,2	0,39	0,35
4 290	1:50	24,0	13,8	53	0,61	1,0	33,2	0,76	0,74
4 290	1:50	24,0	13,8	53	0,61	0,75	33,2	1,30	1,25
4 290	1:50	24,0	13,8	53	0,61	0,5	33,2	2,01	1,84
4 290	1:100	24,0	14,5	53	0,61	1,0	33,2	0,98	0,96

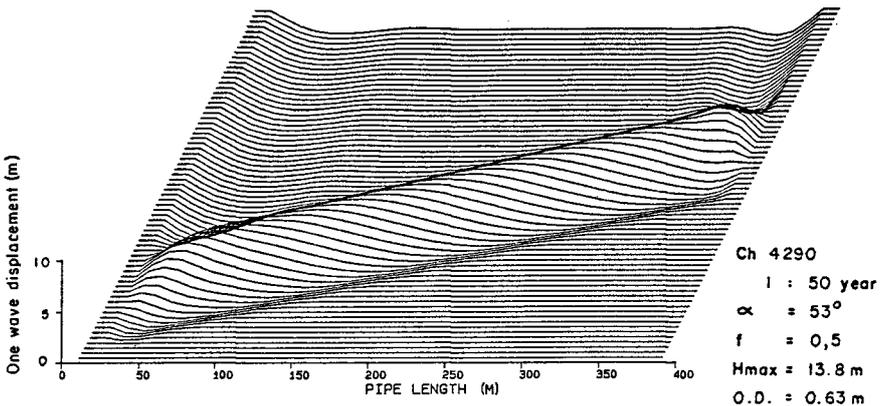


Figure 4: Movement of Pipeline section under Single Design Wave

5. CONCLUSIONS

For a proper pipeline design reliable wave height, period and direction data are needed. For the design of the Richards Bay pipelines a detailed wave refraction analysis was combined with nearshore (Waverider) wave height measurements to determine the design wave conditions along the pipeline route. The choice of the design wave conditions for calculations of the anchor weights depends on the acceptable pipeline movements. For flexible pipelines designed according to the SDP the anchor weighting is based on a 1 in 1 year maximum wave height.

To determine the weighting, the bed kinematics must be calculated using a suitable higher order wave theory and the forces on the pipeline must be calculated using a suitable wave force theory. For the Richards Bay pipelines the bed kinematics were calculated using the Vocoidal wave theory and the wave forces were calculated using the Morison equation.

Pipeline movements should be estimated for the design life of the pipeline using a numerical model. The movements will depend largely on the actual weighting and the resistance of the pipeline with anchors to movement (friction). For the Richards Bay pipelines the movements were calculated using a finite difference model developed by Prof Larsen (Abbot, Larsen and Verwey).

Since no data was available on the friction factors for the Richards Bay pipelines with the star weights and because the assumption of $f = 1$ appeared optimistic, full scale friction tests were done to determine the friction factor for star weights on both hard surfaces ("rock") and on sand. The results showed that the friction factors fell predominantly in the range $f = 0,7$ to $0,8$ and that a value of $f = 0,75$ would seem generally applicable. However, initial friction factors (small movements) on sand can be as low as $f_{0-0,2} = 0,4$ to $0,5$.

Using the Vocoidal wave theory, $f = 0,75$ and the actual weighting, movements were found to range from 1,3 to 1,9 m for 1 in 50 year maximum waves. This movement was considered acceptable except for two rocky reef areas where additional weighting was added to those sections of the pipelines passing through these reef areas.

It became clear from these studies that if movement of the pipeline has to be limited to small values, eg. 0,1 to 0,2 m, as in the case where an HDPE pipeline traverses rocky areas with pinacles, the required weighting may become so large that the pipeline with the anchor weights attached cannot be floated out during laying, thereby losing much of the advantages of using HDPE piping. It must therefore be concluded that the flexible pipeline concept, which allows acceptable movements of the pipeline under design wave conditions, is particularly suitable for locations where the sea bed consists predominantly of sand.

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