

CHAPTER 102

EROSION AROUND A PILE DUE TO CURRENT AND BREAKING WAVES

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ABSTRACT

Tests have been performed on a vertical pile subject to current only and to a combination of current with normal waves and current with breaking waves. The scour around the pile produced by current only is decreased by normal short waves superimposed upon that current and increased when breaking waves are superimposed upon the current. After analysis of the velocity profiles in the undisturbed area upstream of the pile and next to the pile, the following explanation is found for this phenomenon. When normal short waves are superimposed upon a current, the bottom shear stress of the combination of current with waves is increased more in the undisturbed area than next to the pile in the scour area. This results in a decrease of the scour around the pile. Due to the large values of the orbital velocity under breaking waves this effect is reversed for the combination of a current with breaking and relatively long waves. This results in an increase of the scour around the pile.

1. INTRODUCTION

Around piles of jack-up platforms along the Dutch coast in a zone where breaking waves occur occasionally, scour depths have been observed which are significantly more than the normally occurring 1.0 to 1.5 times the pile diameter. Normally the scour around a structure due to a combination of waves with current is less than the scour as a result of current only (Bijker, 1986). In order to try to find an explanation for the unexpected deep scour with a combination of current and breaking waves, tests have been performed in a flume and in a basin of the Laboratory of Fluid Mechanics of the Delft University of Technology. Since there are no bracings between the legs of a jack-up platform and the distance between the legs is large as compared with their diameter, one single pile is studied.

2. TESTS

2.1 General

Three test-series have been performed, with an as much as possible constant mean current velocity, $v = 0.40$ m/s, a water depth, $h = 0.285$ m, and a mean grain size diameter $D_{50} = 0.2$ mm.

Tests of series I have been performed in a flume of 0.8 m width and with a pile diameter $\phi = 0.048$ m.

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Tests of series II have been performed in a wave basin with the wave direction perpendicular to the current direction. The pile diameter was again 0.048 m.

Tests of series III have also been performed in the wave basin, but with a pile diameter $\phi = 0.09$ m.

2.2 Specific conditions

All series have been performed with three conditions, viz.:

- A. Only current, no waves. The water depth varied between 0.283 and 0.296 m and the current velocity between 0.391 and 0.461 m/s.
- B. Current with normal short waves. Current velocity and water depth were as much as possible equal to that for current only. The wave height varied between 0.034 and 0.108 m and the period between 0.97 and 1.23 s.
- C. Current with breaking waves. Current velocity and water depth were again almost equal to those for conditions A and B. The height of the breaking waves varied between 0.190 and 0.214 m with periods between 2.83 and 3.11 s.

In all tests rather uniform sand with a mean grain size $D = 0.2$ mm was used.

All tests are summarized in Table 1. Velocity profiles have been measured in most tests by means of an electromagnetic current meter (made available by Delft Hydraulics) in the undisturbed area upstream of the pile and next to the pile at a distance of 0.02 m from the wall of the pile. The bed form has been measured with a profile-follower.

The tests of series I have been performed in a flume of 25 m length with a width of 0.8 m which at the inflow side was equipped with a generator for regular waves. The sand bed had a thickness of about 0.1 m. The pile was placed at a distance of 10 m downstream of the inflow of the flume. It is assumed that at this place the flow will be completely adjusted. A sketch of the flume is given in Figure 1. From comparison with the tests in the basin it is concluded that the current pattern around the pile is somewhat influenced by the limited width of the flume. The influence is, however, so small that the tests will still demonstrate the differences between the various conditions.

During the tests of this series I the direction of propagation of the waves has been in the flow direction.

The tests of series II and III have been performed in a wave basin which is shown in Figure 2. The current was guided towards the test section by two training walls. In the tests of series II and III the wave direction has been perpendicular to the current direction and the -regular- waves were guided from the wave generator by training walls to the test section. At the end of these training walls the waves will diffract. The influence of this diffraction is, however, negligible at the location of the pile. Opposite to the wave generator a wave damping slope has been installed.

Since the distance between the location of the pile and the upstream side of the actual test section is rather short, sand has been applied on the slope upstream of the test section. This sand has avoided excessive scour of the 0.2 m thick sand layer at the upstream side of the test section and by the roughness it assisted in establishing the equilibrium velocity profile. Measurements of the velocity profile near the pile have indicated that this goal has been reached.

In Figure 3 typical examples of normal short waves and relatively long breaking waves are given. The orbital velocities at the bottom corresponding to short and breaking waves are given in Figure 4. Figure 5 shows the various measuring lines in the wave basin along which depths are measured. The values of the maximum scour are indicated

Table 1

Date	Uz cm/s	Uz norm. cm/s	F'	F''	Uy cm/s	H cm	Cz cm	Cx cm	M cm	T s	Uy cm/s	Uy cm/s	Tcw cm	Sx 10 ⁻² cm ² /s	Sz 10 ⁻² cm ² /s
I A															
	V	4.43	3.85	5.92	3.7	46.1	29.5	6.48					1.88	4.86	4.34
	N	6.56	5.69	21.12	43.5		35.6						4.88	9.78	
I B															
18-11	V	4.42	3.87	5.48	2.4	45.7	28.3	5.88	2.48	1.23	7.46	1.46	2.32	5.34	6.32
	N	5.41	4.73	18.56	58.2	58.2	35.6				6.81	1.19	5.32	18.73	
26-11	V	4.43	3.87	5.91	4.5	45.8	28.3	6.38	6.48	1.23	13.28	2.68	3.44	7.41	11.84
	N	4.93	4.31	7.56	49.9	49.9	35.7				11.48	2.28	5.95	11.89	
I C															
	V	2.65	2.41	1.88	5.8	43.8	29.6	7.88	19.28	3.11	53.88	26.28	9.58	12.34	36.75
	N	3.81	2.75	1.23		45.8	35.9				47.88	23.65	22.46	28.84	
II A															
	V	4.17	4.15	7.59	3.8	48.2	29.6	6.55					1.69	2.66	2.54
	N	4.91	4.79	19.88	37.4	37.4	34.4						2.89	3.49	
II B															
21-1	V	3.45	3.34	2.77	2.2	37.7	27.3	5.88	6.38	1.16	13.78	2.58	3.84	5.68	5.95
	N	5.81	5.31	23.68	35.4	35.4	32.9				9.12	1.76	3.82	4.14	
19/23-1	V	3.45	3.76	4.77	3.4	36.7	27.8	3.88	18.75	1.16	23.88	4.25	5.72	8.75	12.81
	N	3.21	3.48	4.21		36.8	38.6				16.88	3.12	5.17	7.43	
II C															
26-1	V	1.77	1.54	8.81	2.8	46.1	27.4	7.88	19.28	2.83	54.88	24.78	13.58	15.24	35.93
	N	4.32	3.75	4.89		49.4	32.6				39.36	17.76	17.85	21.35	
28/29-1	V	2.95	2.63	8.71	2.5	44.9	29.8	8.18	21.48	2.83	59.38	26.78	14.55	14.89	37.78
	N	5.37	4.79	15.35		44.5	33.7				43.88	19.42	19.47		
III A															
	V	3.89	3.88	6.57	4.8	38.1	28.3	18.15					1.63	2.48	1.88
	N	5.82	5.14	28.99	35.6	35.6	37.8						2.42	2.45	
III B															
12-5	V	4.27	4.39	9.55	2.8	38.9	28.2	8.88	9.28	8.97	16.32	2.47	3.69	6.93	8.48
	N	4.68	4.75	12.59		41.8	35.1				9.84	1.51	4.15	6.66	
14/15-5	V	3.68	3.62	4.15	3.4	39.8	28.3	8.95	9.28	1.16	19.35	2.55	4.87	8.43	12.88
	N	5.44	5.48	25.15		39.3	37.2				11.84	2.16	4.11	6.39	
III C															
14/15-5	V	2.52	2.23	8.38	2.8	45.4	31.8	11.88	15.28	1.88	37.18	11.18	12.32	14.22	36.92
	N	6.16	5.43	15.24		49.6	39.7				28.88	8.67	11.43	17.32	
13-5	V	1.77	1.81	8.85	3.2	38.2	28.5	18.48	21.88	2.81	58.68	26.28	13.94	12.93	31.94
	N	4.35	4.44	7.92		44.7	39.5				38.48	17.28	15.72	17.78	

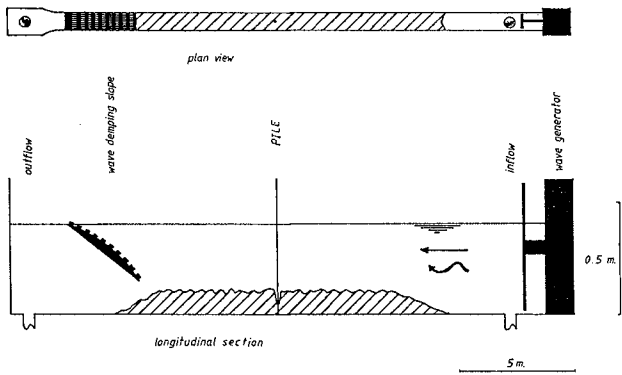


Figure 1
Flume

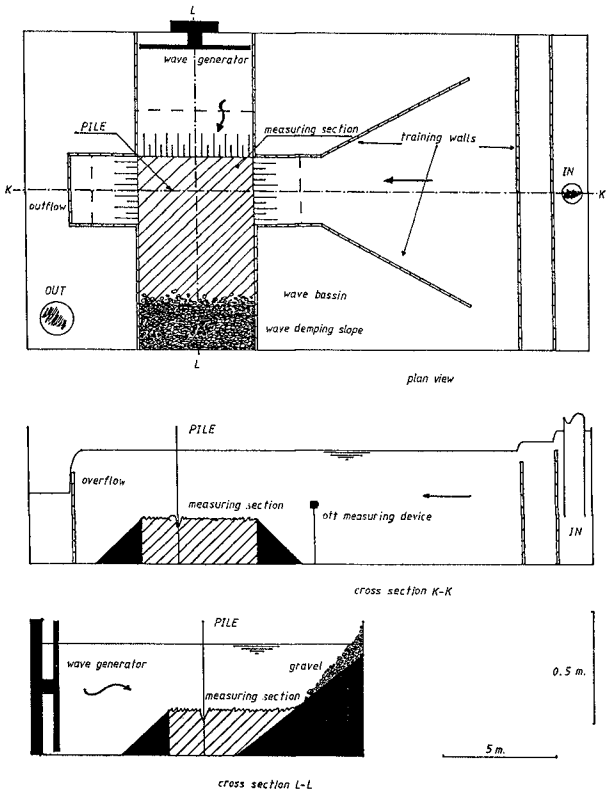


Figure 2
Wave basin

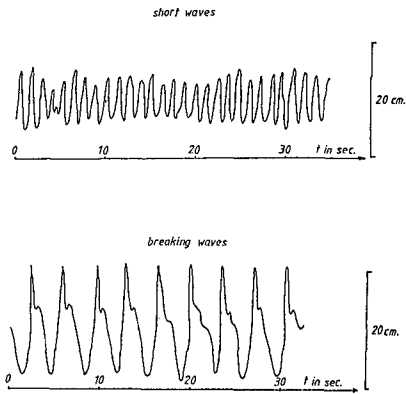


Figure 3
Typical wave profiles

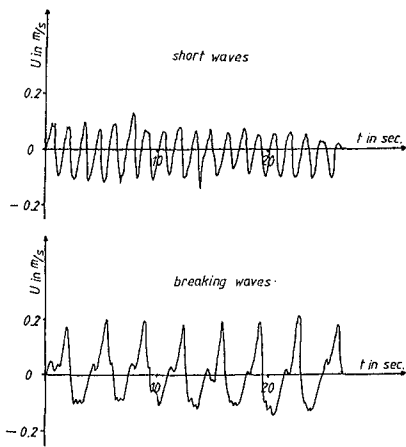


Figure 4
Typical orbital velocity profiles

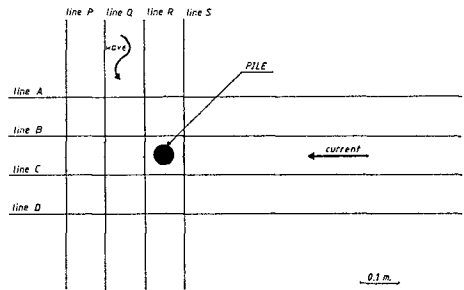


Figure 5
Measuring lines

in Table 1 with the line indication and an asterisk (so deepest point in line C is indicated by C*).

3. TEST RESULTS

3.1 Principles

The principle reason for any scour development around a structure is the difference in transport capacity upstream of the structure and around it. The possible relative increase -or decrease- of suspended load with respect to bed load in the scour area will result in extra -or less- erosion than would result from the change of the value of the total transport (Leeuwestein et al., 1985, and Bijker, 1986). Since, however, in this case the length of the scour area is rather limited, this effect can probably be neglected and the change of the bed load transport, dS_b/dx , will be the determining parameter.

Although in principle a method for the computation of the transport and subsequent scour development around a structure is available (Leeuwestein and Wind, 1984), it was not considered feasible to apply this method here since the transport module in this procedure is not yet fully developed. Also a more general computation of transports upstream and around the structure is not considered for the following reasons.

- The development of the velocity distribution should be measured very detailed and the actual scour computation would require extensive transport computations.
- The current around the pile is accelerating and the effect of an accelerating current on the transport is not yet fully understood.

The main goal in this research is, therefore, to explain the difference in scour development under the various conditions and to define the main parameters which determine this development.

3.2 Scour data

In order to give an impression about the order of magnitude of the transport, the quantities as measured in the wave basin and as calculated for the undisturbed conditions are given in Table 2.

Table 2: Measured and calculated transports

Condition	Measured S_{tot} in m^3/h	Calculated S_{tot} in m^3/h
A	0.05 - 0.10	0.05
B	0.10 - 0.20	0.19
C	0.50 - 1.00	0.54

The transport is calculated by the method suggested by Bijker for the combination of waves and current (Bijker, 1971).

In the ultimate (equilibrium) situation of the scour development the transport upstream of the pile and around the pile must be equal. Since, however, the flow lines around the pile are closer together than upstream of the pile, the transport per unit of width around the pile will be, also in the equilibrium situation, somewhat bigger than in front of the pile in the undisturbed area. The bottom shear stress around the pile must be, therefore, also in the equilibrium situation, bigger than in the undisturbed area.

In the Figures 6 through 8 some typical profiles along line B are shown. Since these profiles resulted from momentary surveys, the maximum scour depths are not necessarily equal to those given in Table 1.

4. DISCUSSION

4.1 Bottom shear stresses

The final scour around a structure will depend, as demonstrated in Chapter 4, on the ratio between the transport upstream of the structure and around it. It is difficult to calculate this transport, but at any rate it will be a function of the bottom shear stress. Therefore, these bottom shear stress values will be compared. As it is not well possible to measure these bottom shear stresses directly, they will be determined from the velocity profiles.

In the Figures 9 through 11 the velocity profiles for the undisturbed flow are given. The bottom shear stress can be calculated from the velocity profile under the assumption that it is that of a completely developed boundary layer. Such a profile can be described according to Prandtl (1926) and von Karman (1930) by

$$v(z) = \frac{v_*}{\kappa} \ln \frac{z}{z_0},$$

with $v_* = \sqrt{\tau/\rho}$ = shear stress velocity, z = height above the bottom, z_0 = height above the bottom, where $v(z)$ is theoretically zero and κ is the constant of von Karman, equal to 0.4.

Since it is difficult to determine exactly the height of a measuring point above a rippled sand bed, the formula is written as

$$v(z) = \frac{v_*}{\kappa} \ln \frac{z' + \Delta z}{z_0' + \Delta z},$$

in which z' is the height above the assumed bottom and Δz is the difference between the assumed and real bottom.

With a least square procedure the value of Δz is chosen in such a way that the difference between the actual profile and the logarithmic profile is minimal. In this way the most reliable value of v_* can be determined.

4.2 Combination of waves and current

The above described method cannot be used in case of a combination of waves and current because the bottom shear stress is determined by the actual velocity, which in this case is fluctuating in magnitude and direction through the orbital motion. This velocity differs from the velocity component in the main flow direction which is measured by the electromagnetic current meter (Bijker, 1968, 1971). The resultant bottom shear stress can be written as: (Bijker, 1986)

$$\tau_{cw} = \tau_c + \frac{1}{2} \tau_w,$$

in which $\tau_w = \frac{1}{2} f_w \rho \hat{u}_b^2$, with f_w = bed friction coefficient according to Jonsson (1966). Swart (1976) has written this factor as

$$f_w = \exp \left[-5.977 + 5.21 (a_b/r) - 0.194 \right],$$

with a_b = amplitude of the orbital motion at the bottom and r = bottom roughness.

This formula holds for $a_b/r > 1.47$. For $a_b/r < 1.47$, f_w remains constant and equal to 0.32.

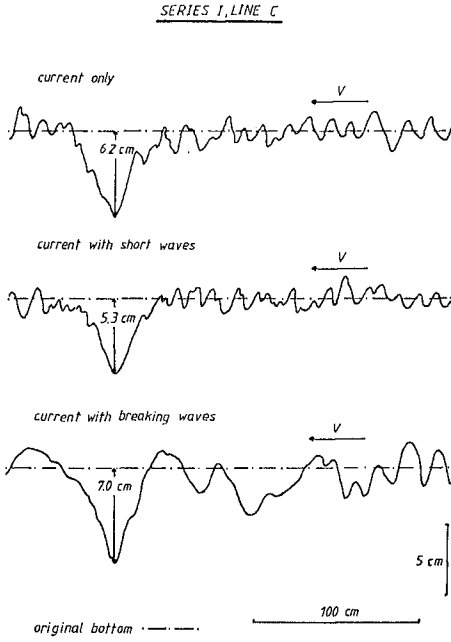


Figure 6
Typical scour profile

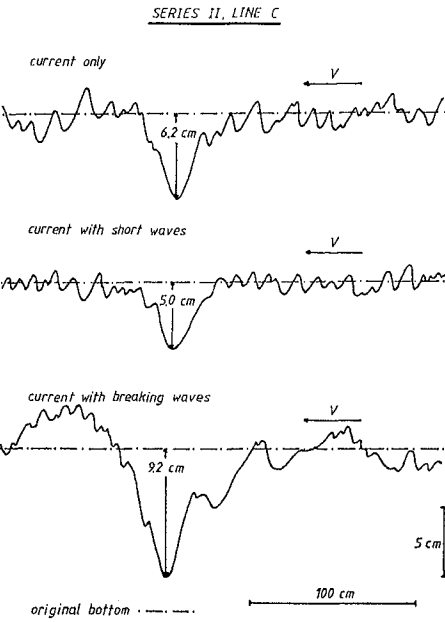


Figure 7
Typical scour profile

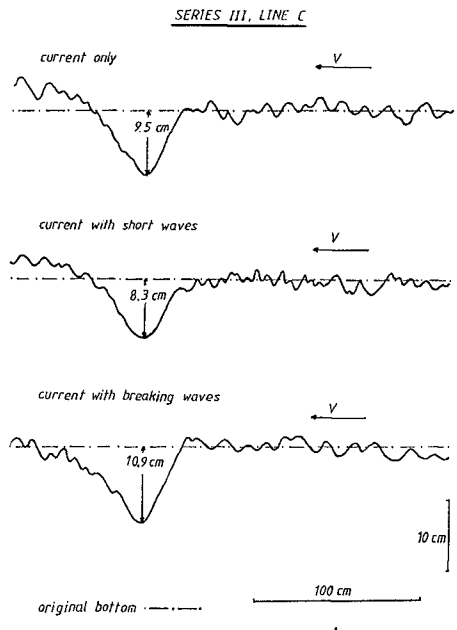


Figure 8
Typical scour profile

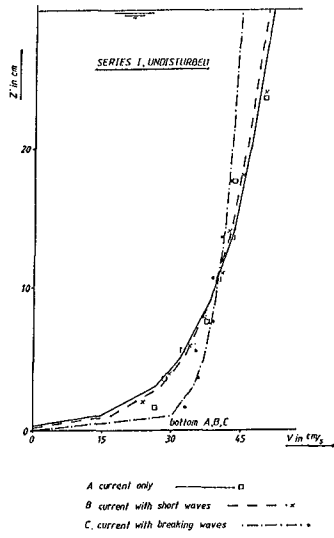


Figure 9
Typical velocity profiles

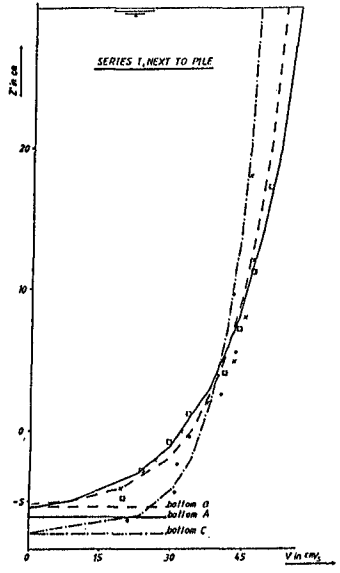


Figure 12

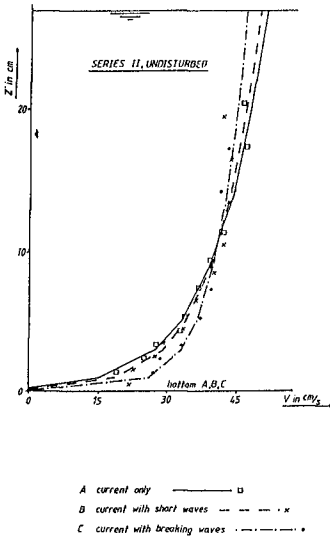


Figure 10

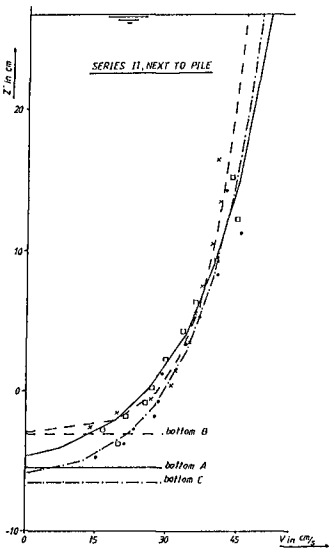


Figure 13

Typical velocity profiles

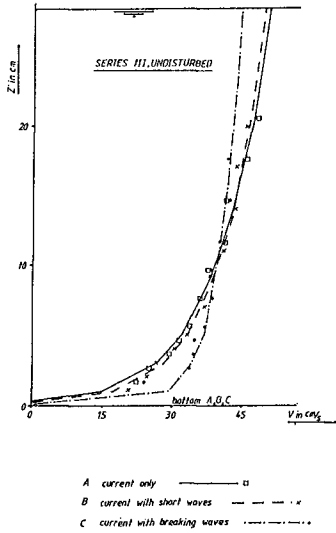


Figure 11

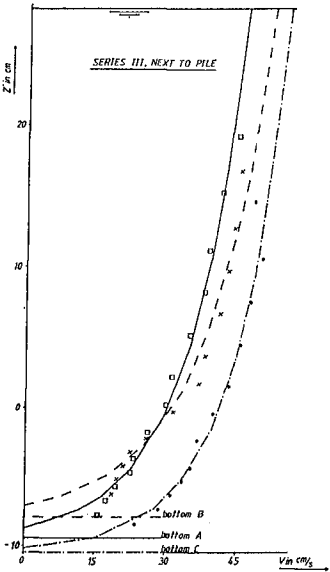


Figure 14

Typical velocity profiles

The values of f_w as function of a_b/r are shown in Figure 15. The value of r can be determined from the velocity profile with current only as $33 z_0$ and is given as r' in Table 1. The value can also be estimated from the ripple size and is given in Table 1 as r'' . The agreement is acceptable, but since the determination of the bottom roughness through the velocity profile is physically the most justified, this value is used as reference. In the flume the average of the calculated values is 0.05 m and in the basin 0.07 m.

4.3 Velocity profiles for the current

The velocity profiles of Figures 9 through 11 are all normalized to a mean velocity of 0.4 m/s. The profiles for current only and for the combination of waves and current are almost equal. There is some steepening of the profile for the combination of waves and current which is most likely the result of the greater value of the turbulent mixing coefficient in the case of a combination of waves and current (Van de Graaff, 1988). This is even more so in the case of breaking waves with much higher orbital velocities. The velocity profiles next to the pile above the scour hole (Figures 12 through 14), which are also normalized to the undisturbed velocity of 0.40 m/s, show the same tendency. However, in series III with the larger pile diameter of 0.09 m, the velocities measured next to the pile are relatively larger than for the smaller pile diameter of 0.048 m in series I and II. The reason is that the velocities are always measured at equal distances (0.02 m) from the pile wall. For the large pile the velocities are measured, therefore, relatively closer to the pile and so they are higher.

4.4 Velocity profiles for orbital motion

The orbital velocities are calculated by the first order linear wave theory and the results are shown, together with the measured values, in Figures 16 and 17 for series II and III with waves and current perpendicular to each other. The measurements next to the pile show an orbital velocity which is approximately 20% less than the calculated value due to the shadow working of the pile. For breaking waves the difference between calculated and measured values is greater.

4.5 Shear stress

From the velocity profiles in the scour hole next to the pile a much higher value is calculated for the apparent bottom roughness. This is caused by the increased turbulence as a result of the deceleration of the current in the scour hole. Therefore, the bottom shear stress is increased. The bottom shear stress of the waves is, however, not increased since this shear stress has already the maximum value due to the low value of a_b/r . This results in a relatively smaller increase of the value of $\tau_{cw} = \tau_c + \frac{1}{2} \tau_w$ in the scour area for the situation with waves and current than for the situation with current only. This leads to a lower value of the equilibrium depth of the scour hole in the case of waves and current than for current only.

In the case of the combination of current with breaking waves, the value of the orbital excursion at the bottom is so high that the maximum value of f_w is not reached. In this case the value of f_w in the scour hole is, therefore, greater than for the undisturbed area. This increase of f_w is just as the increase of τ_c the result of the increased apparent bottom roughness in the scour hole. This results in a relatively higher value of τ_{cw} in the scour hole for the combination of breaking waves and current than for current only.

The various values of the ratio between the bottom shear stress in the scour hole (τ_{cw}/N) and in the undisturbed area (τ_{cw}/V) are shown in Figure 18.

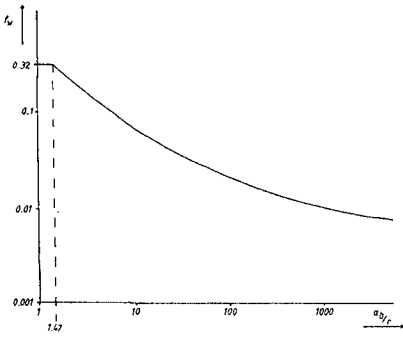
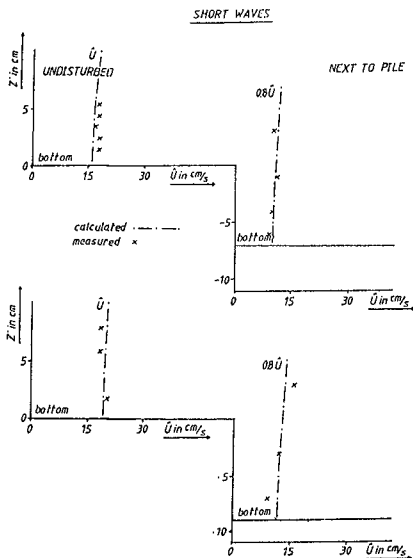


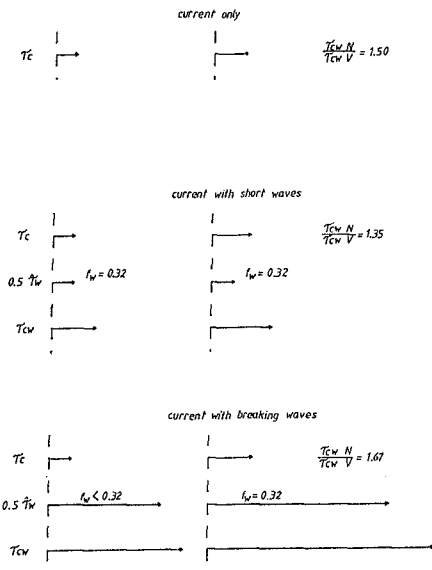
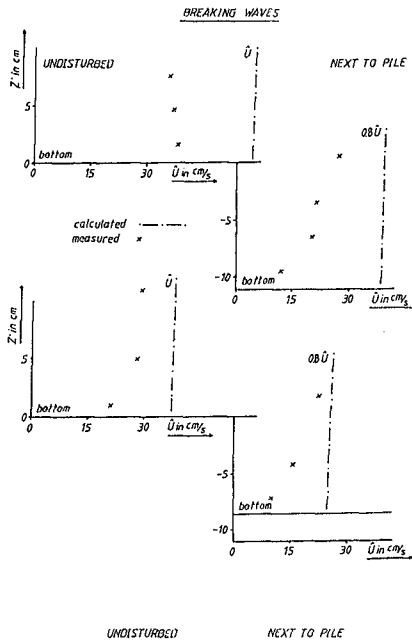
Figure 15
Relation f_w versus a_b/r



Series II

Figure 16
Typical orbital velocity
profiles

Series III



The equilibrium depths of the scour hole have been explained by the effect of the apparent bottom roughness in the scour hole on the bottom shear stress for the situation with current only and for the situation with a combination of current with breaking and non-breaking waves. When the equilibrium depth is not yet reached, the scour is simply the result of the increased velocities around the pile. Only the extent to which the equilibrium scour develops will differ for the various circumstances.

4.6 Comparison with prototype situations

This research has been started because of the observed strong scour around the piles of a jack-up platform placed in -sometimes- breaking waves. The difference between the prototype situation and these tests is caused by the difference in bottom roughness. In front of the pile, in the undisturbed flow, the roughness in the prototype is probably not more than 2 to 3 times the roughness in the model. However, in the scour hole, around the pile, the roughness depends on the depth of the scour hole which is a function of the pile diameter. This diameter will be in the prototype 10 or 20 times that in the tests and the relative increase of the bottom roughness in the scour hole will be, therefore, more in the prototype than in the model. This results in a greater increase of f_w in prototype conditions with breaking waves than in the model. This leads to a greater value of the equilibrium scour depth. In prototype conditions indeed scour depths of 1 to 3 times the pile diameter have been found.

4.7 Conclusions

Normally waves will not increase, but even decrease the scour around a structure as compared with that by current only. The depth of this scour is in the order of 1.5 times the pile diameter. In the case of breaking waves this value can be, however, considerably higher. This may necessitate protective measures where otherwise the scour could be accepted.

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