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Seabed Scour Assessment for Offshore Windfarm

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The prediction of scour at offshore windfarm foundations in areas with mobile seabeds is a challenging topic. In areas with strong currents and wave action, and in areas with shallow water with the additional process of wave breaking to consider, it is necessary to complete laboratory testing. The work described in this paper examined the scour at foundations for a coastal site with waves and strong currents crossing at an oblique angle. The scour in the sand bed was tested for a range of current dominated and extreme wave conditions. The detailed scour profiles were used to determine the depth and extent of scour for a range of water levels, currents and boundary conditions for waves. The testing confirmed the need for scour protection to be installed.

A careful design of the scour protection was required to ensure performance of the foundations. The information on scour depth and extent for the design conditions was used to value engineer the scour protection design. A range of options was tested and the most appropriate one was selected based on a quantified and acceptable level of damage and the degree of interaction of the scour protection design with the surrounding seabed.

Following installation of the offshore wind turbines data was collected on the depth and extent of scour in the field. A reasonable agreement was found between the laboratory results and site observations of the scour hole. This added confidence to the scour protection design that was selected and installed.

The OPTI-PILE design tool gave the correct form of behaviour for rock stability but should be recalibrated for use in such shallow water with such fast currents.

I. INTRODUCTION

The Arklow Bank Wind Park is now operational, with 7 of GE's 3.6MW wind turbines generating clean power for the people of Ireland. Arklow Bank is a shallow water sandbank situated between 10km and 12km offshore from the eastern coast of Ireland and has some of the best wind resources in the British Isles. The project is co-owned by GE and Airtricity, who commissioned Scott Wilson to design the scour protection system for the monopiles[1]. Scott Wilson commissioned HR Wallingford Ltd to perform physical model tests of scour around the monopiles and of the stability of and scour around various scour protection designs[2].

Arklow Bank is subject to overall seabed movement, such as movement of the sandbank, channel migration and overall erosion and accretion. In addition the installation of the monopile foundations for the wind turbines was predicted to (and did) cause local scour around the monopiles. Scour was caused by the strong currents, often over $2ms^{-1}$ that flow over the sandbank and design wave heights that approach 6m on the offshore side (see Section II). The water depth is as low as 5m over the crest of the bank so depth-limited wave breaking occurs during severe storms.

The prediction of local scour at offshore windfarm foundations in areas like Arklow Bank with mobile seabeds is a challenging topic that requires laboratory testing to optimize the design. This paper describes two sets of physical model tests performed by HR Wallingford with Scott Wilson to assist in the scour protection design.

The first set of tests comprised five scour tests of the sand bed around the 5m diameter monopile. These tests were aimed at determining the maximum depth and extent of scour around the monopile under extreme conditions and confirmed the need for scour protection to be installed.

A careful design of the scour protection was required to ensure performance of the foundations. Therefore a second set of ten tests was conducted with scour protection in place to assess the damage to the scour protection and the scour at the edge of the scour protection. Details of the scour protection designs are presented in Section VI. The most appropriate option was selected based on a quantified and acceptable level of damage and the degree of interaction of the scour protection design with the surrounding seabed.

Following installation of the offshore wind turbines data was collected on the depth and extent of scour in the field and was compared to the experiments in Section X.

After the completion of these tests, HR Wallingford derived the OPTI-PILE design tool [3] which aids the preliminary design of scour protection works for offshore monopile wind turbines located on a sandy seabed. The OPTI-PILE design tool was calibrated against a different set of laboratory tests and has been verified against field data from Scroby Sands Offshore Wind Farm [4]. It has been applied to the Arklow tests in Section XI.

II. SCALING OF THE PHYSICAL MODEL

Practical considerations led to the choice of a geometric length scale of 1:36 for the physical model. The hydrodynamics were scaled using Froude scaling [5] so velocities were a factor of 6 lower in model than prototype. However, the situation is complicated by the need to model the mobility of the scour protection rock and sand, and the flow around the monopile.

The wave-induced scour around circular cylinders depends on the Keulegan-Carpenter, KC, number [6]. This occurs as the Keulegan-Carpenter number represents the way in which the wave flow interacts with the monopile. Froude scaling preserves the Keulegan-Carpenter number for the monopile.

If Froude scaling is applied to the rock protection also, the Keulegan-Carpenter number of the rock will be scaled correctly and hence the pressure gradient term for the rock will also be scaled correctly.

A. Scaling of sand

The mobility of sand and rock is commonly determined using its Shields Parameter, which is the ratio of the force exerted by the bed shear stress on the grain to the submerged weight of the grain counteracting this [7]. The Shields parameter, θ, is defined as:

$$\theta = \frac{\tau}{g(\rho_s - \rho)d} \tag{1}$$

Where τ is the bed shear stress (Nm^{-2}), g is gravitational acceleration (ms^{-2}) ρ_s is the density of the sediment (kgm^{-3}), ρ is the density of the water (kgm^{-3}) and d is the sediment diameter. The wave skin friction shear stress, τ_w, is given by:

$$\tau_w = \tfrac{1}{2}\rho f_w U_m^2 \tag{2}$$

Where f_w is the wave friction factor and U_m is the near-bed wave orbital velocity (ms^{-1}). This friction factor is a function of U_mT/d for rough turbulent flows. As Froude scaling gives the same values for U_mT/d in model and prototype, (where T is the wave period) it follows that the wave friction factor will be the same in model and prototype, providing both are rough turbulent flows.

The limit on grain size and rock, for which the same model and prototype Shields parameter (due to wave skin friction) can be obtained from Froude scaling, is at the transition from rough turbulent to smooth-turbulent flow. The transition was obtained from Equations 62a and 63 of [7] and was found to be around 0.05mm for typical model values.

However, a scaled version of the Arklow Bank sand would have been so small it would have been a cohesive sediment and would not behave like a sand. Therefore a larger, but fine, washed sand, was used in the model. In this case Redhill 110 with median grain diameter 0.11mm was used.

The interaction of the rock protection and surrounding seabed in terms of scour will still be reproduced in the model if the same sediment transport regime exists in the model and prototype. In order for sediment to remain in suspension its fall velocity, w_s, must be smaller than the turbulent component of velocity which is related to the skin friction shear velocity, $u_{sts}=(\tau_w/\rho)^{0.5}$. Therefore, the relative skin friction velocity = u_{sfs}/w_s should both be greater than one in model and prototype, for suspended sediment. Preliminary results calculated using linear wave theory and equations in [7] indicate that both model and prototype storm conditions will produce suspended sediment transport.

An alternative criterion for suspended sediment transport [8] was also met in model and prototype storm conditions. Therefore the mechanism of sand transport will be similar in model and prototype so the interaction of the rock protection and surrounding seabed in terms of scour will be reproduced in the model, but not at the same rate. The model does not, however, reproduce geotechnical effects.

B. Scaling of rock

The rock used in the physical model had a density of $2710kgm^{-3}$, while the prototype rock had a density of $2650kgm^{-3}$. The fluid used in hydraulic model tests was fresh water, the prototype sea water had a greater density. These variations in densities mean that, without compensation, the rock in the model would be more stable than in the prototype. Such a model would underestimate movement and, hence, the damage that might occur. The size of the rock to be used in the model was corrected using the Hudson equation, so that it exhibited the same stability characteristics as the prototype.

III. Physical Model

The physical model was constructed in a wave basin and consisted of:

- Wave paddle to generate irregular waves with the required spectral properties;
- Eastern and western wave flumes, with bathymetry representative of eastern and western sides of Arklow Bank, starting at the -14mCD contour. These ensured that there was realistic wave shoaling and breaking in the model;
- Sand bed with enough space for monopiles and scour protection schemes;
- Model monopile, constructed in two sections: the lower section was attached to the bed of the wave basin and extended just above the sand bed, while the upper section extended above the water surface;
- Wave absorber for the wave flume not in use;
- Re-circulating current input with control structures to widen and steady the flow and ensure that there was a smooth transition onto the sand bed; and
- Sediment trap and sump area where sediment transported from the model can settle out and the current can be re-circulated from.

The monopile was installed in the sand bed in front of the centre of the wave flume to be used in testing (east or west). Testing was performed in one wave flume at a time as the flumes required different wave conditions. A wave absorber was installed in the flume not being used as otherwise waves entered the sand bed at different times from the different flumes as the shoaling was different.

Wave heights were measured in front of the wave paddles and at the end of the moulded bathymetry, before the waves entered the sand bed. The eastern moulded bathymetry had its high point before the sand bed as the modeled monopile was to be on the western side of the Bank's crest. Currents were measured using a current meter just before they entered the sand bed.

Bed profiles were measured along 8 radial lines from the monopile suing a touch-sensitive bed profiler. The top section of the monopile was removed and the profiler head (shown in Fig. 1) was positioned on the edge of the monopile to provide a known starting locations. Measurements were made every 15mm from there.

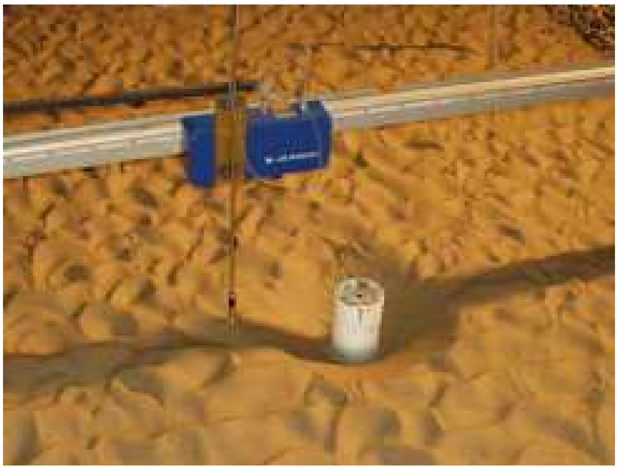

Figure 1. Bed profiler measuring scour hole

The layout of the test section is shown in Fig. 2, which also shows the numbering scheme adopted for the profiling lines. Dimensions in Fig. 2 are given in model scale. Results given in the paper are presented at full or prototype scale.

IV. HYDRODYNAMIC CONDITIONS

A design current speed of 2.3ms^{-1} was specified for all tests. Wave conditions with return periods of 50 years and 200 years were provided by Scott Wilson at the 14m CD contour on the east side and west side of Arklow Bank, as shown in Table I. The wave conditions in the western flume were calibrated until the measured wave conditions at the -14mCD contour matched the target conditions. The wave heights in the eastern flume were increased until the point where increasing the wave height generated failed to increase the measured wave height at the sand bed due to depth-limited wave breaking over the crest of the sandbank, which was offshore from the monopile location.

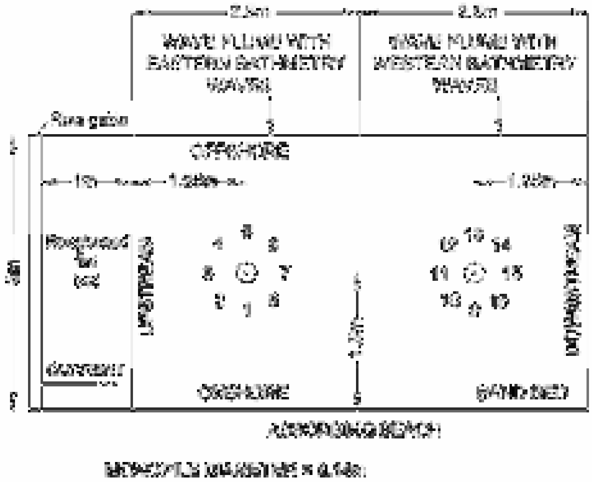

Figure 3. Sketch of test section showing current and wave flumes, sand bed, monopile positions and profiling line numbers

	Bank side	Wave return period [years]	Hs [m]	Tm [s]	Tp [s]	Water depth [m]	Current speed [ms^{-1}]
A	East	50	5.1	10.2	13.1	8.2	2.3
B	East	200	5.8	10.7	13.7	10.2	2.3
C	West	50	3.2	10.3	13.2	8.2	2.3
D	West	200	3.9	10.7	13.7	10.2	2.3

TABLE I. TARGET HYDRODYNAMIC CONDITIONS

V. SCOUR TESTS

Five tests were carried out of scour around a monopile: a current only test with the current speed, U_{cr}, just above the threshold of motion, then four wave plus current tests representing 50- and 200-year return period conditions from the east and west (conditions A to D in Table I). The threshold current test was run for 24 prototype hours, while the wave plus current tests were run for 6 prototype hours.

The sand bed was screeded flat and eight radial profiles of the initial sand bed level were measured before the start of each test. The current was started and the current speed checked using a current meter, before starting the waves (in tests 2 to 5). Scour depths around the monopile were measured using a metre stick throughout the tests. At the end of the test the waves and currents were stopped, the water was drained from the basin, photographs were taken and the same 8 radial lines were re-profiled.

The sand bed at the end of the first, current only, test is shown in Fig. 1. The time-development of the scour during this test is shown in Fig. 4 at prototype scale. A fitting routine that minimised the mean absolute difference between the measurements and the curve was used to determine values for the equilibrium scour depth and timescale of scour to a curve of the following form:

$$S(t) = S_{eq}\left(1 - e^{-t/T}\right) \qquad (3)$$

where S(t) is the scour depth at time t, S_{eq} is the equilibrium scour depth and T is the timescale of the scour. The average value of S_{eq} = 4.8m and the average value of T = 2.4 hours. The maximum scour depth for any of the points was close to one monopile diameter.

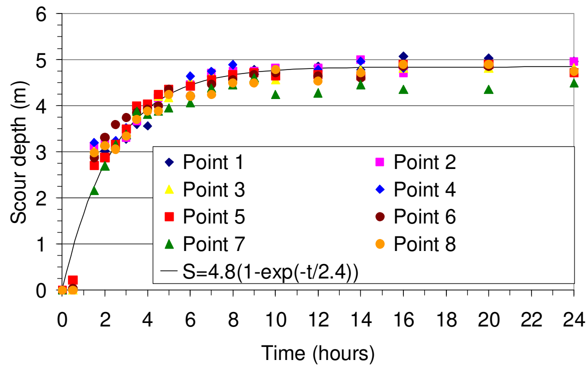

Figure 4. Time development of scour from current-only test

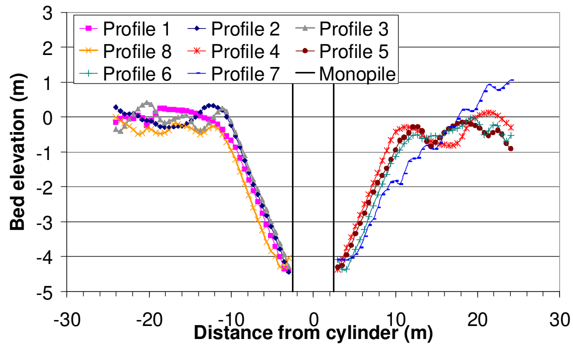

Figure 5. Radial bed profiles after sand bed Test 1

The 8 radial bed profiles measured at the end of Sand Bed Test 1 are shown in Figure 5, with zero in the vertical being the original bed level. Measurements made before the test showed that the original bed was typically within 0.04m (prototype) of zero. Figure 5 shows that the sides of the scour pit generally had a fairly constant bed slope of around 30°to 32°.

The same procedure was repeated for the four wave and current tests (sand bed Tests 2 to 5) except that the 200-year return period condition tests followed on from the bathymetry left by the 50-year return period tests. Therefore the bed was screeded flat after sand bed Test 1, 3 and 5, but not after Tests 2 and 4.

The resulting equilibrium scour depths, S_{eq} and timescales, T are presented with their standard deviations, σ_S and σ_T respectively in Table II., which shows that all the sand bed scour tests produced substantial scour pits. The scour pits had smooth, almost conical walls with side slopes of around 30° to 32°. The base of the scour pit was generally flatter and the side slopes were generally less steep at the top, in the region where the scour pit started to interact with the bedforms that covered the whole of the sand bed.

Test 1 was conducted because current-only scour tests have been observed to give the greatest scour depth, in tests using a smaller diameter pile. That was not the case here with the greatest scour depths coming from wave & current Tests 2 and 3, which gave a relative scour depth of S_{eq}/D = 1.4, where D is the monopile diameter. The largest measured scour depth was 6.5m.

Test 1 was also run for longer than the other tests (24 prototype hours) to investigate the timescale of the scouring. Test 1 showed that conducting a test for six (prototype) hours will amount to approximately 2.5 times the timescale so the scour will be just over 90% of the equilibrium scour depth.

Test	Cond-ition	S_{eq} [m]	σ_S [m]	T (min)	σ_T (min)
1	Ucr	4.8	0.1	143	16
2	A	7	0.5	230	7
3	B	7	0.4	178	30
4	C	6.3	0.4	155	52
5	D	5.9	0.2	88	33

TABLE II. EQUILIBRIUM SCOUR DEPTHS AND TIMESCALES OF SCOUR WITH STANDARD DEVIATIONS FOR SAND BED TESTS

Although the timescale has been converted to a prototype equivalent value using Froude scaling, in practice the timescale in the field will be different (probably shorter) as the sand in the field under storm is relatively smaller and more mobile, with a higher ratio of mean shields parameter over threshold shields parameter.

VI. SCOUR PROTECTION TEST SETUP

The ten scour protection model tests were carried out using different combinations of four scour protection schemes and three environmental conditions. The four scour protection designs are described below and their cross-sections are provided in Fig. 6.

A. Two layers of armour stone over 1m deep filter layer in a 20m wide ring around the monopile (plus 1:2 side slopes). The prototype armour rock weight was in the range 1 tonne to 3 tonnes and the filter layer prototype median weight was 200kg. More detailed specifications for armour rock and filter material are given in Table III. A geotextile was placed under the 200kg rock.

B. Two layers of armour stone over 1m deep filter layer in a 20m wide ring around the monopile (plus 1:2 side slopes). The prototype armour rock weight was in the range 0.3 tonne to 1 tonne and the filter layer prototype median weight was 65kg. More detailed specifications for the armour stone and filter material are given in Table IV. A geotextile was placed under the 65kg rock.

C. One metre deep filter layer in a 10m wide ring around the monopile (plus 1:2 side slopes) in an excavated pit so that the top of the filter layer was flush with the top of the sand bed. The filter layer prototype median weight was 200kg (as used in design A). No geotextile was used.

D. Two layers of armour stone (1 tonne to 3 tonnes) over 1m deep filter layer, sunk into the sand bed so that the top of the filter layer was flush with the top of the sand bed. There was a 10m wide ring of filter and armour around the monopile. The filter layer had a 1:2 slope at the outer edge. The prototype armour rock weight was in the range 1 tonne to 3 tonnes and the filter layer prototype median weight was 200kg (as for design A). No geotextile was used.

The three environmental conditions were:

i. 50-year return period wave and water level conditions for the western side of Arklow Bank, plus 2.3m/s depth-averaged current, (i.e. Hs = 3.2m, Tm = 10.3s, water level = +2.8m CD.

ii. 200-year return period wave and water level conditions for the western side of Arklow Bank, plus 2.3m/s depth-averaged current, (i.e. Hs = 3.9m, Tm = 10.7s, water level = +4.8m CD.

iii. Low water condition with depth limited waves for the western side of Arklow Bank, plus 2.3m/s depth-averaged current, (i.e. Hs = 3.0m, Tm = 10.3s, water level = -1.4m CD.

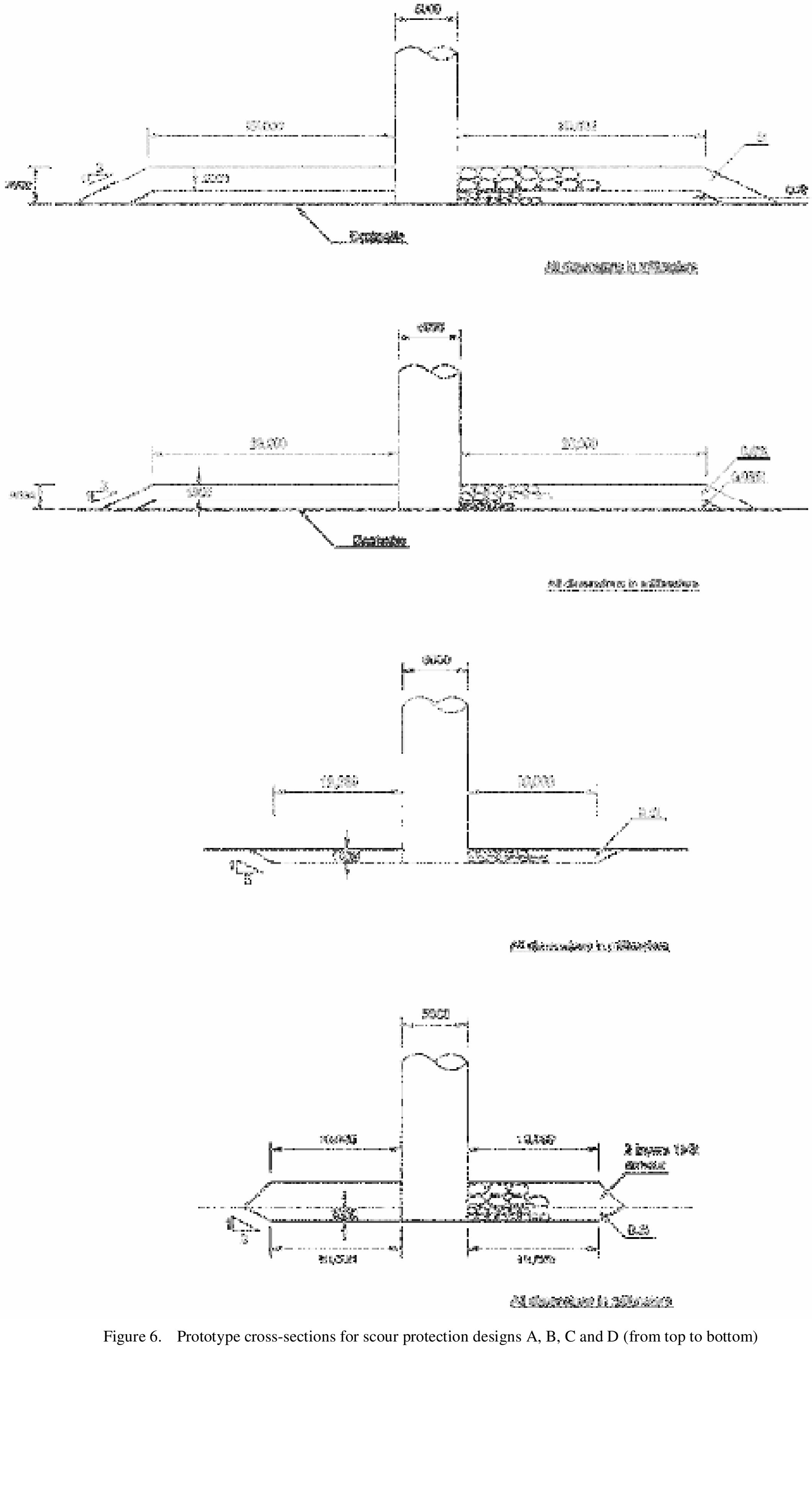

Figure 6.   Prototype cross-sections for scour protection designs A, B, C and D (from top to bottom)

	Armour stone		Filter layer	
% finer	Prototype mass (tonne)	Model Mass (kg)	Prototype mass (tonne)	Model mass (kg)
0	0.93	0.016	0.09	0.002
10	1.08	0.019		
25	1.36	0.023	0.14	0.0031
50	2	0.034	0.2	0.0045
75	2.93	0.051	0.29	0.0064
90	3.69	0.064		
100	4.3	0.074	0.43	0.0095

TABLE IV. SCOUR PROTECTION DESIGN A, C AND D ARMOUR STONE AND FILTER WEIGHTS

	Armour stone		Filter layer	
% finer	Prototype mass (tonne)	Model Mass (kg)	Prototype mass (tonne)	Model mass (kg)
0	0.28	0.005	0.028	0.0006
10	0.33	0.006		
25	0.43	0.007	0.043	0.001
50	0.65	0.011	0.065	0.0015
75	0.99	0.017	0.099	0.0022
90	1.27	0.022		
100	1.51	0.026	0.151	0.0033

TABLE III. SCOUR PROTECTION DESIGN B ARMOUR STONE AND FILTER WEIGHTS

The ten scour protection (SP) tests were model representations of:

SP Test 1. Scour protection design ‘A’ with environmental condition ‘i’ (50-year return period conditions). The test was run from a flat bed for six hours.

SP Test 2. Scour protection design ‘A’ with environmental condition ‘ii’ (200-year return period conditions). The test was run from the final bathymetry of the previous test, for six hours.

SP Test 3. Scour protection design ‘A’ with environmental condition ‘iii’ (low water conditions). The scour protection was rebuilt on a flat bed. The test was run for six hours.

SP Test 4. Scour protection design ‘B’ with environmental condition ‘i’ (50-year return period conditions). The scour protection was built on a flat bed and the test was run for six hours.

SP Test 5. Scour protection design ‘B’ with environmental condition ‘i’ (50-year return period conditions). The test was run from the final bathymetry of the previous test, for thirty hours.

SP Test 6. Scour protection design ‘B’ with environmental condition ‘iii’ (low water conditions). The scour protection was rebuilt on a flat bed and the test was run for six hours.

SP Test 7. Scour protection design ‘C’ with environmental condition ‘iii’ (low water conditions). The scour protection was built starting with a flat bed and the test was run for six hours.

SP Test 8. Scour protection design ‘D’ with environmental condition ‘iii’ (low water conditions). The scour protection was built starting with a flat bed and the test was run for six hours.

SP Test 9. Scour protection design ‘D’ with environmental condition ‘iii’ (low water conditions). The test was run from the final bathymetry of the previous test for thirty hours.

SP Test 10. Scour protection design ‘C’ with environmental condition ‘ii’ (200-year return period conditions). The scour protection was built starting with a flat bed and the test was run for six hours.

VII. RESULTS OF SCOUR PROTECTION TEST 1

Overhead photographs of Scour Protection Test 1 are shown in Fig. 7. Note that the top part of the monopile has been removed and that the camera is at an angle to the flow. The brick wall on the bottom, left hand side of the photograph is the downstream edge of the sand bed.

The damage caused during Scour Protection Test One can be seen by comparing the two photographs in Fig. 7. Fig. 7 shows that the sand bed is rippled throughout (so sediment transport is occurring throughout) with the ripple

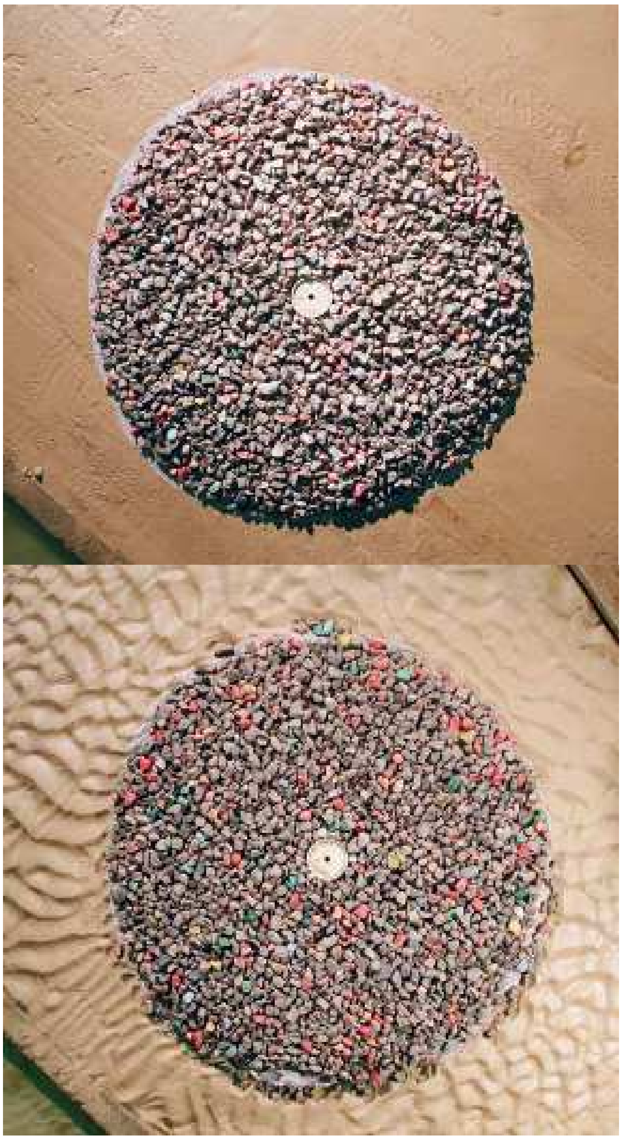

Figure 7. Before (top) and after (below) Scour Protection Test 1

crest direction predominately perpendicular to the current direction on the upstream and onshore sides of the model (implying development is current dominated). The ripple patterns are more three-dimensional on the offshore side, showing the influence of waves. There is a small area downstream of the model where the ripple crests are parallel to the incident wave crests showing that their growth is wave dominated.

Bed profiles at the end of the test are shown in Fig. 7 (top). The scour (negative values) and deposition (positive values) were calculated by subtracting initial profiles from final profiles and are shown in Fig. 7 (below) for Scour Protection Test 1. Fig. 7 shows that there were some changes in elevation between the edge of the monopile and displacement of stones (at 2.5m radial distance) and about 15m radial distance. This may have been due to flow acceleration around the monopile. Note however, that changes in elevation between 'before' and 'after' may occur due to slightly different displacement of the probe in the two deployments or to minor rotations or displacements of armour stones. Consistent changes over two or three points are a more reliable indication of movement than differences at a single point.

The profile showing the greatest changes in elevation close to the monopile is profile 13, which points directly offshore (into the waves). It is therefore likely that the environmental conditions could cause some stone movement on the flat top of the model, close to the monopile. Little movement can be detected from the changes in bed elevation at the edge of the protection, shown in Fig. 8. However, the photographs (Fig. 9) reveal that some stones have moved out from the edge of the model, off the geotextile.

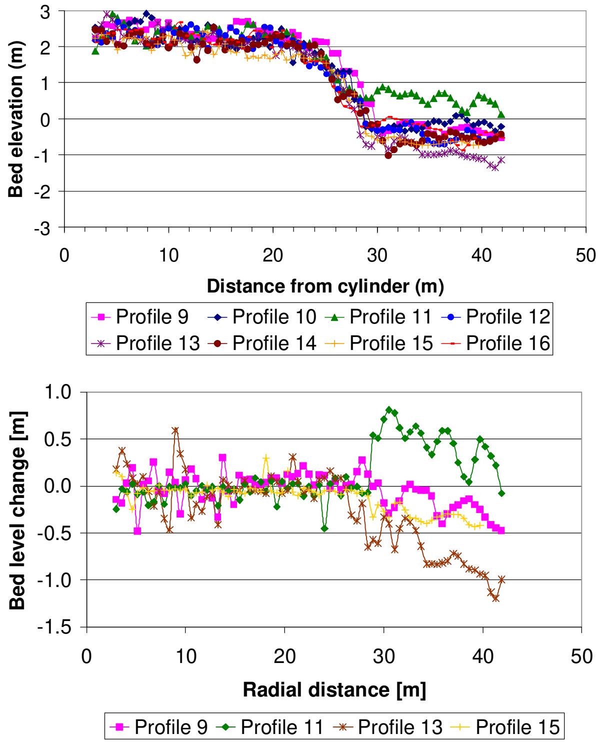

Figure 9. Bed levels at the end of Scour Protection Test 1 (top) and bed level changes during the test (below)

VIII. Results From Other Scour Protection Tests

Scour Protection Test 1 was presented in some detail to show the type of data collected for all the scour protection tests. Examples are given below from the other scour protection tests. Fig. 8 (top) shows scour protection design B before SP Test 4, while Fig. 8 (below) shows the same design at the end of SP Test 5, after 36 hours of environmental condition ii.

Two rows of armour stones were added to the downstream edge of the model before Scour Protection Test 5 was run which minimized the depth of the scour pit that had formed at the hard edge of the sand bed during SP Test 4.

The photographs in Fig. 8 reveal that some stones have moved out from the edge of the model, off the geotextile. Where this has occurred, there is often a strip of geotextile visible between the remaining stones in the model and those that have slipped down and out from the edge of the model. This movement is associated with bed lowering and shows that the model is unraveling by movement away from the model. The radial profiles showed that steep slopes typically occur between 23m and 25m radial distance, with a flatter area where the displaced stones have settled between 25m and 27m radial distance.

Equivalent sets of photographs for SP Design C are shown in Fig. 9, before and after SP Test 10. Here the upper surface was spray-painted in concentric rings so that

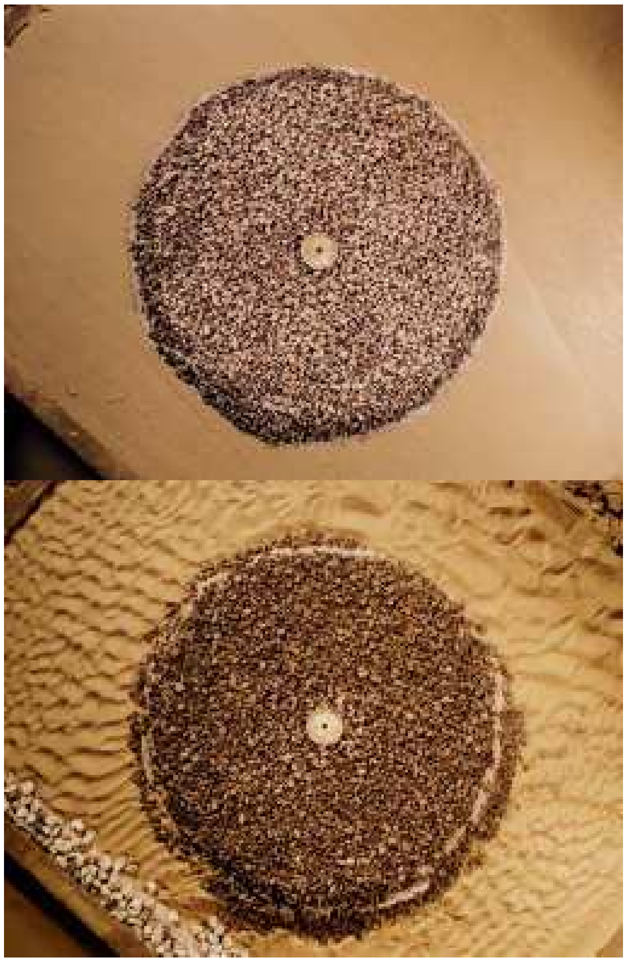

Figure 8. Scour protection Design B before SP Test 4 (top) and after SP Test 5 (below)

Figure 10. Scour Protection Design C before (top) and after (below) SP Test 10.

movement could be assessed at different distances from the monopile. Fig. 9 shows that there was relatively little movement of scour protection material during the test, although some occurred along profile 11 close to the monopile and some occurred at the edge of the model, where the seabed lowered. Fig 10 shows overhead photographs of Design D after SP Test 9.

IX. DAMAGE ANALYSIS

Damage was defined as the numbers of armour stones that were displaced by more than one diameter, expressed as a percentage of twice the total number of armour stones visible in the top layer of armour (as the models had two nominal armour layers). Damage calculations were made by printing 'before' and 'after' overhead photographs of the scour protection system onto acetates and overlaying them. This method allows the stones that had moved to be identified, marked and counted.

Damage was calculated for each of the following four quadrants:

Figure 11. Model after Scour Protection Test 9

- scour profile 9 (beach) to scour profile 11 (upstream)
- scour profile 11 (upstream) to scour profile 13 (offshore)
- scour profile 13 (offshore) to scour profile 15 (downstream)
- scour profile 15 (downstream) to scour profile 9 (beach).

For breakwaters and similar marine structures to breakwaters, accepted practice within the industry is to consider damage to the whole structure of between 2% and 5% as 'onset of failure' and damage between 6% and 10% as 'failure' [9]. These figures are for stone movement of more than one diameter. Ref [9] recommends that tests are run for six hours at design conditions, as here.

Damage statistics from the Scour Protection Tests with larger armour stones are given in Table V, which shows that most damage occurred in the two offshore quadrants between profile lines 11 and 15 (Fig. 2). Designs A, B and C all showed damage of over 10% in one of the offshore quadrants, although SP Tests 5 and 9 were unusually long (at 30 hours each). Therefore the structure as a whole may be said to be on the onset of failure. However, two factors should be borne in mind:

1. Damage was predominantly limited to the edge of the structure, well away from the monopile that the scour protection was designed to protect. The scour protection suffered an insignificant amount of damage close to the monopile.
2. Damage mainly took the form of armour stones moving down and out from the edge of the armour in locations where the bed lowered around the protection. The armour stone is expected to reach a stable condition in time, through the formation of a falling apron, even though the maximum lowering is about 3m.

Scour protection Design C, which used the filter layer from Design A as its main armour was not assessed in the same way as the stones were too small. It was used in SP Tests 7 and 10. Bed level changes from the bed profiling were used to assess damage here. Erosion of up to 0.5m was measured right against the monopile in SP Test 7, which otherwise showed relatively low changes in bed level. The maximum erosion close to the monopile in Sp Test 8 was less than 0.25m and again there were relatively small changes in the sand bed level around the model, with a maximum lowering of just under 1m immediately downstream (profile 15).

Test	**Design**	**Lines 9-11**	**Lines 11-13**	**Lines 13-15**	**Lines 15-9**
One	A	1	3.4	5.1	0.5
Two	A	0.3	1.4	2.4	0.3
1+2	A	1.3	4.8	7.5	0.8
Three	A	3.2	9.8	12.2	2.5
Four	B	1.5	1.9	3.3	2
Five	B	4.4	9.3	8.1	5.6
4+5	B	5.9	11.2	11.4	7.6
Six	B	3.2	7.9	7.4	2.3
Eight	D	0	0	0	0.6
Nine	D	0	11	2	0
8+9	D	0	11	2	0.6

TABLE V. DAMAGE STATISTICS FROM DIFFERENT QUADRANTS

X. Installation and Observed Scour

Installation of the initial 7 wind turbines was achieved in only 9 weeks during late summer and early autumn in 2003 using a jackup barge fitted with a 1,200 tonne crane [10] as shown in Fig 11. There was a short delay between installation of the monopiles and the scour protection, which was sufficient for scour holes to develop around the monopiles, due to the tidal current alone.

Side scan sonar was used to measure the size of the scour holes and an example of a contour plot derived from side-scan sonar is shown in Fig. 12. The scour hole is fairly symmetrical, with smooth sides and is about 4m deep. It has a similar form to the scour hole measured in the laboratory current-only test, shown in Fig. 1.

Scour protection was installed using a back-hoe on the side of a jackup barge. The scour protection design, as built, was based on Design C, but included filling up the naturally-occurring scour hole, so extended deeper than shown in Fig. 5, providing a greater level of scour protection.

XI. Application of OPTI-PILE Spreadsheet

The OPTI-PILE design tool is a spreadsheet that calculates the main parameters needed for a conceptual scour protection design for an offshore monopile wind turbine on a sandy seabed [3]. The spreadsheet calculates the depth, extent and volume of the predicted scour hole. The stable rock size for scour protection, the extent of protection and the mass of rock required can also be calculated for both a static design and a dynamic design.

A number of key parameters were calibrated using a set of physical model tests of the scour protection around an offshore wind turbine in the Dutch sector of the Southern North Sea. The OPTI-PILE design tool was subsequently verified [4] against field data obtained from the Scroby Sands Offshore Wind Farm, off the East Anglian coast.

The conditions at Arklow are rather different from those used to calibrate the OPTI-PILE design tool: the current speed is over twice as fast, the wave heights are just over half the size and the water is less than half the depth. The comparison between the predicted and observed rock stability will indicate whether the OPTI-PILE design tool can be used outside the original limits of its calibration.

Figure 12. Installation of wind turbines at Arklow. © Airtricity

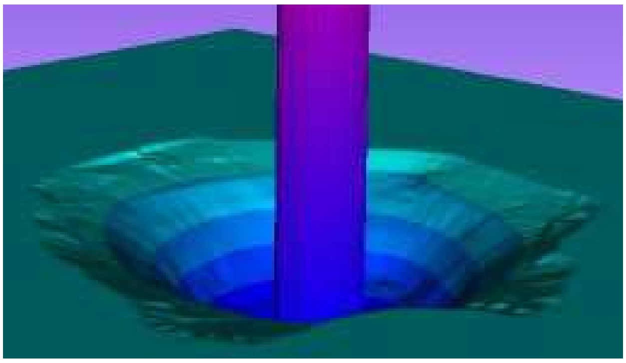

Figure 13. Scour hole measured at Arklow between installation of monopile and scour protection

The OPTI-PILE design tool calculates a stability parameter, $Stab = \theta_{max} / \theta_{cr}$ where θ_{max} is the maximum Shields parameter and θ_{cr} is the critical Shields parameter for the inception of motion. The value θ_{cr} =0.056 (for stone diameters greater than 0.01m) was taken in conjunction with the maximum bed shear stress. Higher values of Stab are less stable. Critical values of Stab were set based on the experimental tests. The test results were classified into the following three categories:

1) No movement of rocks;
2) Some movement of rocks, but not sufficient to cause failure;
3) Failure.

Here, different failure criteria were used so the results are not directly comparable to the damage statistics given in Table V. The failure criteria were:

- Static scour protection has failed when a section of top layer armour has disappeared completely, exposing the filter layer over a minimum area of four armour units;
- Dynamic scour protection has failed when a volume of rock has eroded that is equivalent to the volume of rock that had to be eroded for a static failure.

Table VI lists the ten scour protection tests with the scour protection design and environmental condition used. The OPTI-PILE stability parameter, Stab, is then given with the damage category the stability parameter falls into. Three values for measured damage are then given:

1) Maximum damage from Table V;
2) E_{max}/d_{50}, where E_{max} is the maximum erosion measured using the bed profiler over the flat top of the model (excluding lowering at the edges) and d_{50} is the median armour diameter;
3) E_{max}/AD where AD is the armour depth from Figure 6.

Table VI shows that the stability parameter, Stab is not closely related to the maximum damage. It is related to the damage statistics E_{max}/d_{50} and E_{max}/AD, as shown in Figure 14, which also shows best-fit straight lines through the data. These lines were used to predict damage from Stab to obtain a relative mean absolute error of 0.19 in both measures of damage.

The OPTI-PILE design tool stability number increases with increasing measured damage: the correct behaviour for a stability predictor. However, the different measures of damage used in [3] and here have made the

SP Test	Design	Environment	STAB	Oti-Pile Category	Max Damage (%)	E_{max}/d_{50}	E_{max}/AD
1	A	i	0.277	N	5.1	0.44	0.24
2	A	ii	0.301	N	2.4	0.49	0.27
3	A	iii	0.426	S	12.2	0.64	0.35
4	B	i	0.302	N	3.3	0.74	0.42
5	B	i	0.302	N	9.3	0.40	0.23
6	B	iii	0.461	F	7.9	0.77	0.44
7	C	iii	0.493	F		0.96	0.48
8	D	iii	0.426	S	0.6	0.37	0.20
9	D	iii	0.426	S	11	0.60	0.33
10	C	ii	0.35	N		0.44	0.22

comparison more difficult. The Arklow experiments only measured changes in the armour level along 8 radial profiles and it would be possible for greater levels of damage to occur in between. The definition of failure for a static model used in OPTI-PILE implied a maximum erosion, $E_{max} = 1$, a condition that was not observed in the tests. The OPTI-PILE design tool would need to be recalibrated for design use in similar circumstances.

XII. SUMMARY AND CONCLUSIONS

A set of laboratory experiments were conducted to investigate scour depths and the stability of scour protection around a monopile foundation in relatively shallow water (sometimes less than the monopile diameter) with high current speeds and breaking waves.

The scour tests showed that equilibrium scour depths of up to 7.5m (1.5 pile diameters) were possible under the wave plus current conditions tested. The timescale of scour in the model indicated that 80% to 90% of the equilibrium scour could occur in the course of six hours. The scour pits produced had smooth side slopes of around 30°, which were elongated in the downstream direction.

The flow acceleration around scour protection designs ‘A’ and ‘B’ (Fig. 6) which stood 3m and 2m above the bed caused substantial lowering of the sand bed around them. This trend was particularly pronounced at low water condition, when the initial water depth was only 4m and bed lowering of up to 3m was observed. Although there is some evidence of armour stone movement on the flat tops of these models (particularly close to the monopile where there would have been higher than ambient shear stresses due to local flow accelerations) the majority of the damage occurred at the edge of the model, where the bed had lowered.

The percentage damage determined in the tests were high enough to be considered as the onset of failure or as failure itself, according to the criteria [9] for coastal breakwaters. However, the scour protection designs tested are a different type of structure, which will not collapse if a few stones are moved in the interior or at the edges, but

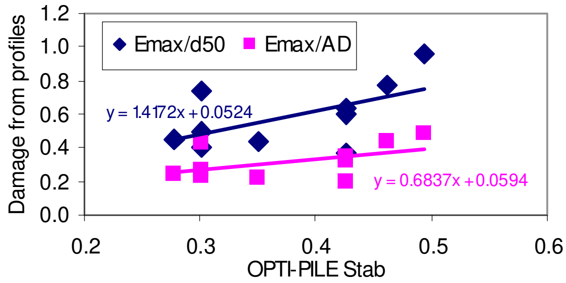

Figure 14. Measured damage against predicted damage

which will gradually unravel at the edges. A higher percentage damage can therefore be tolerated.

The results for scour protection design ‘D’ (Fig. 6) were of a similar form to the results from design ‘A’ and ‘B’. The lower structure caused less flow acceleration and bed lowering around it. Damage levels were correspondingly lower as well (compare the results from Test 8 with those from tests 3 and 6 which used the same environmental conditions with designs A and B).

The two tests with design ‘C’ (Fig. 6) showed little apparent damage, although only filter material was used. This design had its top surface flush with the bed, so it did not disrupt the flow.

The OPTI-PILE design tool was used outside its calibrated range and gave the correct form of behaviour for a stability protector but should be re-calibrated for use at such low depth and high current speeds.

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