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Laboratory Tests of Scour at a Seawall

J. Sutherland*, C. Obhrai*, R.J.S. Whitehouse* and A.M.C. Pearce**
* HR Wallingford Ltd., Wallingford, Oxfordshire, OX10 8BA, UK
** University of Southampton, Southampton, SO17 1BJ, UK

A set of medium-scale laboratory tests of wave-induced scour at seawalls has been performed in a flume at HR Wallingford. The methodology is presented along with test conditions and summarized results. The scour depth at the toe of the seawall is highly dependent on the form of wave breaking onto the structure. Sea states where waves plunge directly onto the wall generate jets of water that may penetrate to the seabed and cause a local scour hole immediately adjacent to the seawall. This is a different scouring mechanism to that observed in deeper water and is also absent when the seawall is well within the surf zone and most of the large waves have broken before they reach the seawall. Theoretical limitations are discussed.

I. INTRODUCTION

Toe scour is believed to be the most common cause of seawall failure in the UK [1] yet some of the empirical predictors of toe scour derived from laboratory tests give results with opposite dependence on relative depth [2, 3, 4]. Moreover many of the existing laboratory experiments on scour in front of seawalls were performed at relatively small scales, leading to tests dominated by bedload transport [2]. The pattern of sediment erosion and accretion varies with the mode of sediment transport, so bedload transport gives a different bed profile to suspended load transport [2, 5].

In many circumstances scour problems in the field will be dominated by suspended sediment transport. The results from physical model tests are likely to be misleading unless the most important scaling parameters are satisfied. Therefore, laboratory tests should be done with a justification of the scale used and with an awareness of the ambiguities that have arisen in previous experiments done at small-scale. As a minimum it is recommended that the dominant transport mode (bedload or suspended) is reproduced in the laboratory. In many cases when considering sand beaches this will require suspended sediment transport to be produced in the laboratory for a significant proportion of the time. This will require experiments to be performed at a medium to large scale.

A number of wave flume experiments on toe scour at seawalls that were conducted at medium to large-scale, used fine sand and were conducted with irregular waves have been identified and their results collated. This database has been extended and gaps in it filled by devising and performing a set of medium-scale experiments of toe scour in front of smooth seawalls in a new wave flume in the Froude Modelling Hall at HR Wallingford. The laboratory tests looked at the longshore-uniform case of normal wave incidence only.

II. PREDICTION OF SUSPENSION

Scaling rules have been developed for bedload [6] and suspended load [7] tests. In cases where suspended sediment is predominant the Shields parameter does not have to be preserved [7, 8] as the wave breaking and turbulence are more dominant mechanisms in determining sediment mobility than the wave shear stress. In this case an undistorted model with Froude scaling using the same value of the Dean fall speed parameter in model and prototype is recommended. The Dean fall speed parameter, $D_{ws}$, is given by:

$$ D_{ws} = \frac{H_s}{w_s T_p} $$

(1)

Here $H_s$ is the significant wave height, $w_s$ is the fall speed of the median sediment and $T_p$ is the spectral peak wave period. Various formulae have been derived that distinguish between suspended and bedload hydrodynamic regimes. The following criterion has been proposed [9] for the initiation of suspension under waves:

$$ \frac{U_w - U_{cr}}{w_s} \geq 16.5 $$

(2)

where $U_w$ is the maximum value of orbital velocity at the bed and $U_{cr}$ is the critical velocity for incipient sediment transport.

Simple Froude scaling rules exist for the fall velocities of both small and large particles (if the same density material is used in model and prototype) but not for intermediate values. Therefore, Equation 3 [10] was used to determine the sediment fall speed, $w_s$ as:

$$ w_s = \frac{\nu}{d} \left[ (0.36^2 + 1.049D_s^3)^{1/2} - 0.36 \right] $$

(3)

Where $D_s$ is the dimensionless grain size given by:

$$ D_s = \left[ \frac{g(s-1)}{\nu^2} \right]^{1/3} d. $$

(4)

Here $\nu$ is the kinematic viscosity, $d$ is the grain diameter, $g$ is gravitational acceleration and $s = \rho_s / \rho$ is the relative density with $\rho_s$ the sediment density and $\rho$ the water density.

[1] Funded by UK Department of Environment, Fisheries and Rural Affairs (Defra) under R&D project FD1927
The critical velocity for incipient transport, $U_c$, [11] is given by Equation 5:

$$U_c = 0.015 < 0.025$$

The maximum near-bed wave velocity, $U_m$, can be calculated from linear theory for a representative wave. Substituting $U_m$, $U_c$, and $w_b$ back into Equation 2 allows one to determine if a test should generate suspended sediment transport.

**III. EXISTING DATASETS**

Results from the tests reported here are intended to be comparable to those from the following datasets on toe scour that had suspended sediment transport and were conducted with irregular waves.

Xie [9] included a number of tests that generated scour in a flat bed in front of a vertical seawall that were in the suspension mode. Most of the tests used regular waves but three irregular wave tests were also conducted in suspension mode. Scour profiles were provided from two of those tests.

Fowler [3] performed mid-scale (wave heights between 0.2 and 0.3m) laboratory tests of the scouring of a 1:15 sloping sand bed in front of a vertical wall, which was always placed close to the intersection of the beach and mean water line. All 18 irregular wave tests were conducted in the range of 0.011 < $h/L_o$ < 0.025 and 0.015 < $H/L_o$ < 0.040 where $h_b$ is the initial water depth at the toe of the seawall and $L_o$ is the linear theory deep water wavelength. Ottawa sand, with $d_10$ = 0.13mm and a specific gravity of 2.65 was used in all cases.

The SUPER TANK Data Collection Project was performed at the O.H. Hinsdale Wave Research Laboratory (WRL), Oregon State University [12, 13]. The WRL flume is 104.2m long, 3.66m wide and 4.57m (15 feet) deep. The beach was made of very well-graded sand trucked from the Oregon coast with a median diameter, $d_50$ = 0.22mm. Waves were run in relatively short bursts to provide data for numerical model calibration. Only in one toe scour test, ST_CO, were there about 3,000 waves of the same spectrum run. Test ST_CO was the only one directly comparable to these tests.

**IV. EXPERIMENTAL SETUP**

The new tests were performed in a 45m long wave flume at HR Wallingford [14, 15]. The internal cross-section of the flume is 1.2m wide by 1.7m high. Waves were generated using a piston-type wavemaker with a maximum stroke of ±0.6m and a maximum operating depth of 1.4m. The wavemaker has an absorption system for absorbing wave energy reflected from the seawalls. The test setup had a 19.2m long 1:30 smooth concrete slope from the flume floor up to an elevation of 0.64m. The test section was a 5.14m long sand bed filled with Redhill 110 sand. The sand bed was 0.3m deep at the offshore end. Tests 1 to 14 all started from a screeded 1:30 slope. The sand bed level at the wall was therefore approximately 0.80m above the flume floor. Tests 15-34 started from a screeded 1:75 slope where the sand bed level at the wall was approximately 0.7m above the flume floor.

Redhill 110 sand has typically 98.80% SiO$_2$, 0.09% Fe$_2$O$_3$, 0.21% Al$_2$O$_3$ and 0.14% loss on ignition. The results of a sieve analysis of Redhill 110 are shown in Table I. Assuming $d_10$ is the sieve size that n percent of the sand by weight passes through, common percentiles are $d_10 = 0.087mm, d_50 = 0.111mm$ and $d_84 = 0.154mm$. Settling velocities are also given using (3) and (4) for fresh water (salinity = 0) at a typical temperature in the tests of 11°C, giving calculated density of water $\rho = 999.5kgm^{-3}$ and a kinematic viscosity $\nu = 1.272\times10^{-6}m^2s^{-1}$. In addition, a sediment density $\rho_s = 2650kgm^{-3}$ was assumed – appropriate for silica sand.

Waves were measured by 10 wave gauges. A group of 4 wave gauges was situated over the flat flume bed before the start of the beach slope and were used to separate out incident and reflected wave spectra using a least-squares technique. A second group of 4 wave gauges was situated at the offshore end of the sand bed to separate out incident and reflected wave spectra over the sand bed. Variations in the standard deviation in the surface elevation at these gauges could also be used to provide information on the partial standing wave pattern in front of the seawall. A further two wave gauges were placed 1.00m and 0.10m in front of the seawall, where local variations in the water level were greater due to the stronger partial standing wave pattern.

The profile of the sand bed was measured using the touch sensitive, 2D bed profiling system developed at HR Wallingford. The profiler consists of a probe which can move up and down and is mounted on a carriage which moves horizontally along a support beam. On the bottom of the probe is a sensor that consists of a lightweight “finger” that can move freely up and down inside a 20mm cylinder. The cylinder rests either on a small ball that is pulled along the bed by a lever arm mounted on the probe or on a foot that takes small steps (see Fig. 1).

The resolution of the bed profiler was ±1mm in the horizontal direction and ±0.5mm in the vertical direction. However even though the finger is very light it did cause a slight deformation (approximately 1-2mm) of the sand bed. This meant that the profiler tended to smooth out some of the finer features of the bed. Bed profiles were interpolated onto a regular spacing of 1mm.

Each test started from an initial slope of either 1:30 or 1:75 which was achieved using wooden templates installed on each side of the flume. The initial profile was then measured using the bed profiler.

**TABLE I. SIEVE ANALYSIS OF REDHILL 110 SAND**

<table>
<thead>
<tr>
<th>Percent by weight passing sieve (%)</th>
<th>(mm)</th>
<th>(Phi)</th>
<th>Soulsby w, (ms$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.064</td>
<td>3.97</td>
<td>0.0026</td>
</tr>
<tr>
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<td>0.074</td>
<td>3.75</td>
<td>0.0035</td>
</tr>
<tr>
<td>16</td>
<td>0.087</td>
<td>3.53</td>
<td>0.0048</td>
</tr>
<tr>
<td>25</td>
<td>0.095</td>
<td>3.40</td>
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<td>50</td>
<td>0.111</td>
<td>3.17</td>
<td>0.0077</td>
</tr>
<tr>
<td>75</td>
<td>0.135</td>
<td>2.89</td>
<td>0.0111</td>
</tr>
<tr>
<td>84</td>
<td>0.154</td>
<td>2.70</td>
<td>0.0141</td>
</tr>
<tr>
<td>90</td>
<td>0.167</td>
<td>2.58</td>
<td>0.0163</td>
</tr>
<tr>
<td>95</td>
<td>0.177</td>
<td>2.50</td>
<td>0.0180</td>
</tr>
</tbody>
</table>
V. TEST CONDITIONS

A total of 34 tests were performed. Details of the test conditions are given in Table II.

- Test type is either V = vertical, or S = sloping (1:2) with T added if test had varying water level;
- $H_s$ is the measured incident offshore significant wave height (m) calculated by the least squares technique using gauges 1-4;
- $T_p$ is the measured spectral peak wave period (s);
- $h_t$ is the initial water depth at the structure toe (m);
- $C_r$ is the bulk reflection coefficient;
- $S_i$ is the scour depth at the toe of the seawall (m) with negative values being accretion;
- $S_{max}$ is the maximum scour depth (m);
- $X_{sm}$ is the distance from the toe of the structure to the location of $S_{max}$ (m);
- $A_{max}$ is the maximum increase in bed level (m);
- $X_{am}$ is the distance from the toe of the structure to $A_{max}$ (m).

19 tests were performed with a vertical wall. 13 of these were with a beach slope of 1:30 and 6 were with a beach slope of 1:75. Details of these are given in Table 2. A total of 6 scour protection tests were carried out, but are not reported here. A sloping wall at 1:2 was used in 9 of the tests with a beach slope of 1:75.

The majority of tests used a constant incident significant wave height, period and depth to measure the time development of scour. In these tests a bed profile was taken before the test and after 300, 1000, and 3000 spectral peak wave periods.

In the second type (tests 10, 17 and 34a) the water depth was varied to simulate part of a tidal cycle and in these tests a new profile was taken after each of the 300 wave bursts. Test 10 started with a water depth at the wall close to zero, increasing the depth in steps to a maximum depth of 0.3m then decreasing the depth in steps down to -0.1m at the seawall. However test 17 started from a higher water depth of 0.2m and decreased the depth in steps down to -0.05m, while in test 34a the water was slowly drained from the flume during the test.

VI. BED LEVEL CHANGES FOR A VERTICAL WALL

Bed level changes (final – initial elevation) at the end of Tests 7, 12, 4 and 11 (i.e. after 3,000 waves) are provided in Fig. 2. All the tests were performed with a vertical seawall. Negative values represent scour, while positive values represent accretion. These four tests had the same initial bed profile, wave period ($T_p=3.24$ s) and incident wave height ($H_s=0.2$ m) but different water depths ($h_t=0.0\text{m}, 0.1\text{m}, 0.2\text{m}$ and $0.4\text{m}$ respectively). A comparison has been drawn between these four tests as they resulted in very different breaking wave conditions at the wall and hence different bed profiles.

During test 7 ($h_t=0.0\text{m}$) the waves broke offshore and the wave energy was largely dissipated before the waves reached the wall in the swash zone. As a result there was a slight accretion at the wall but a general lowering throughout the rest of the profile. The vertical seawall was situated within the surf zone during Test 12 ($h_t=0.1\text{m}$) and some breaking occurred onto it, although most of the larger waves had already broken by the time they reached the seawall. The resulting scour profile includes a small dip at the toe of the seawall caused by turbulence and a deeper scour hole at about 0.5m from the structure toe.

However during Test 4 ($h_t=0.2\text{m}$) the waves tended to break onto the structure and the impacts sent water high up above the seawall (see Fig. 3). In these cases water plunging down the face of the seawall to the bed, resulted in suspended sediment transport at the toe, and this mechanism generated the deepest scour depths. Fig. 2 shows that the maximum scour occurred at the wall ($0.158\text{m}$) with significant accretion ($0.056\text{m}$) occurring 1.3m offshore.

In deeper water (test 11, $h_t=0.4\text{m}$) the waves did not break onto the seawall as plunging breakers, but tended to reflect more energy. The scouring pattern (shown in Fig 2) in these cases was closer to the classic Xie type standing wave pattern. The maximum scour of $0.117\text{m}$ occurred away from the wall and was significantly less than the plunging breaker case shown in Fig. 2 ($0.158\text{m}$).
Figure 3. Wave breaking against a vertical wall. Metre stick gives scale.

Table II. Details of experiments performed

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Test type</th>
<th>Beach slope</th>
<th>$H_s$ [m]</th>
<th>$T_p$ (s)</th>
<th>$h$ [m]</th>
<th>$C_r$ [-]</th>
<th>$S_t$ [m]</th>
<th>$S_{max}$ [m]</th>
<th>$X_{am}$ [m]</th>
<th>$A_{max}$ [m]</th>
<th>$X_{am}$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>V</td>
<td>1:30</td>
<td>0.193</td>
<td>1.55</td>
<td>0.200</td>
<td>0.50</td>
<td>0.057</td>
<td>0.057</td>
<td>0.031</td>
<td>0.006</td>
<td>0.660</td>
</tr>
<tr>
<td>2</td>
<td>V</td>
<td>1:30</td>
<td>0.193</td>
<td>1.87</td>
<td>0.200</td>
<td>0.49</td>
<td>0.065</td>
<td>0.065</td>
<td>0.031</td>
<td>0.023</td>
<td>0.680</td>
</tr>
<tr>
<td>3</td>
<td>V</td>
<td>1:30</td>
<td>0.198</td>
<td>2.29</td>
<td>0.200</td>
<td>0.47</td>
<td>0.130</td>
<td>0.130</td>
<td>0.031</td>
<td>0.044</td>
<td>0.950</td>
</tr>
<tr>
<td>4</td>
<td>V</td>
<td>1:30</td>
<td>0.194</td>
<td>3.24</td>
<td>0.200</td>
<td>0.46</td>
<td>0.158</td>
<td>0.158</td>
<td>0.031</td>
<td>0.056</td>
<td>1.369</td>
</tr>
<tr>
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<td>V</td>
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<td>0.197</td>
<td>4.58</td>
<td>0.200</td>
<td>0.44</td>
<td>0.140</td>
<td>0.143</td>
<td>0.049</td>
<td>0.073</td>
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<tr>
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<td>V</td>
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<td>0.204</td>
<td>1.87</td>
<td>0.000</td>
<td>0.09</td>
<td>-0.031</td>
<td>0.025</td>
<td>0.731</td>
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<td>7</td>
<td>V</td>
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<td>0.196</td>
<td>3.24</td>
<td>0.000</td>
<td>0.13</td>
<td>-0.011</td>
<td>0.032</td>
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<td>0.197</td>
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<td>0.26</td>
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<tr>
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<td>1.87</td>
<td>-0.1 to 0.2</td>
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<td>V</td>
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<td>0.280</td>
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<td>0.49</td>
<td>0.036</td>
<td>0.052</td>
<td>0.354</td>
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<td>1.87</td>
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<td>0.200</td>
<td>0.31</td>
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<td>0.068</td>
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<td>1.87</td>
<td>0.000</td>
<td>0.07</td>
<td>-0.001</td>
<td>0.010</td>
<td>2.480</td>
<td>0.015</td>
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<td>0.206</td>
<td>3.24</td>
<td>0.000</td>
<td>0.12</td>
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<td>1.87</td>
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<td>0.50</td>
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<td>0.1 to -0.1</td>
<td>-</td>
<td>0.074</td>
<td>0.081</td>
<td>0.210</td>
<td>0.002</td>
<td>2.642</td>
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Figure 2. Bed level changes after tests 4, 7, 11 and 12
VII. EFFECT OF BEACH SLOPE

Four pairs of tests were performed where the same incident wave height, wave period, water depth and structure were used but in the first case the initial beach profile was at 1:30 while in the second case the initial beach slope was 1:75. The four pairs of tests were:

- 2 and 15 ($H_i=0.195m$, $T_p=1.87s$ and $h_i=0.20m$);
- 4 and 16 ($H_i=0.195m$, $T_p=3.24s$ and $h_i=0.20m$);
- 5 and 18 ($H_i=0.194m$, $T_p=4.58s$ and $h_i=0.20m$);
- 11 and 19 ($H_i=0.216m$, $T_p=3.24s$ and $h_i=0.40m$).

The scour depths for the 1:75 beach slope are plotted against the scour depths for the 1:30 beach in Fig. 4, which shows the toe scour depths and the maximum scour depths. It is clear that the 1:75 beach slope gave much lower scour depths than the 1:30 beach slope. Best-fit straight lines through the origin gave toe scour depths in the 1:75 beach as 52% of those in the 1:30 beach, while maximum scour depths were 75% of those in the 1:30 beach. The reason for this is believed to lie in the way the waves broke on the beach in front of the seawall.

The waves tended to break as spilling breakers on the 1:75 beach, where for the 1:30 case there were more breakers plunging onto the seawall causing turbulent jets to reach the seabed, resulting in scour.

VIII. BED LEVEL CHANGES FOR A SLOPING WALL

Bed level changes (final – initial elevation) at the end of Tests 32, 27 and 30 (i.e. after 3,000 peak wave periods) are provided in Fig. 5. All the tests were performed with a 1:2 (V:H) smooth sloping seawall. Negative values represent scour, while positive values represent accretion. These three tests had the same initial bed profile, wave period ($T_p=3.24s$) and similar offshore incident wave height ($h_i=0.19m$ to 0.24m) but different water depths ($h_i=0.0m$, 0.2m and 0.4m respectively). A comparison has been drawn between these three tests as they resulted in very different breaking wave conditions at the wall and hence different bed profiles.

The distance from the seawall is taken as the cross-shore distance from the point where the seawall emerged from the initial screeded bed profile. Test 32, with an initial toe depth of 0.0m again showed accretion. The largest scour depth again occurred for a toe depth of 0.2m (Test 27) while the scour depth for the higher toe depth of 0.4m (Test 30) was again lowered. In each of the latter two cases the initial part of the bed profile follows the line of the sloping seawall. Minor differences between the bed level changes from Tests 27 and 30 where both tests measured the sloping seawall are due to minor differences in the initial bed levels and to some small sand ripples that were left on the seawall at the end of the tests.

Three tests were performed with an incident significant wave height of 0.2m, peak period of 3.24s and water depth at the toe of 0.20m. These were Test 4 (vertical wall, 1:30 slope, $S_i=0.158m$, $S_{max}=0.158m$), Test 16 (vertical wall, 1:75 slope, $S_i=0.089m$, $S_{max}=0.102$) and Test 27 (sloping wall, 1:75 slope, $S_i=0.104m$, $S_{max}=0.105m$). This shows that reducing the initial beach slope reduced the toe and maximum scour depths. Changing the seawall from a vertical to a 1:2 slope increased the scour depths slightly. The differences between using a sloping and vertical seawall are investigated further in the next section.

IX. VARIATION IN SCOUR DEPTH WITH WALL SLOPE

Four pairs of tests were performed where the same incident wave height, wave period and water depth were used but in the first case a vertical wall was present while in the second case a sloping wall was present. In all cases an initial bed slope of 1:75 was used. The four pairs of tests were:

- 15 and 26 ($H_i=0.20m$, $T_p=1.87s$ and $h_i=0.20m$);
- 16 and 27 ($H_i=0.20m$, $T_p=3.24s$ and $h_i=0.20m$);
- 14 and 29 ($H_i=0.26m$, $T_p=1.87s$ and $h_i=0.30m$);
- 19 and 30 ($H_i=0.23m$, $T_p=3.24s$ and $h_i=0.40m$).

The sloping wall scour depth is plotted against the vertical wall scour depth in Fig. 6, for the scour depth at the structure toe and the maximum scour depth. The diagonal line plotted is the line of equivalence. Fig. 6 shows that for the four cases tested the scour depths were not, on average, reduced by replacing a vertical seawall with a 1:2 sloping seawall. This runs contrary to many
people’s expectations [1] that reducing the wall slope reduces the scour depth as it reduces the reflection coefficient.

As with a vertical wall the scour depth reached partly depends on the way the wave runs down the seawall slope and interacts with the following wave. Deep scour depths appear to correlate well with wave run-down reaching the structure toe, which continued as a 1:2 slope under the beach.

X. Variation in Scour Depth with Relative Water Depth

Fig. 7 shows the variation of relative scour depth, $S/H_s$, with relative toe depth, $h_t/L_p$ where $L_p = gT_p^2/(2\pi)$ is the deep water linear theory wavelength for the wave peak period, $T_p$ at the toe depth $h_t$. Fig. 7 (top) shows the scour depth at the toe, $S_t/H_s$, while Fig. 7 (below) shows the relative maximum scour depth, $S_{\text{max}}/H_s$. Fig. 7 shows the highest relative scour depth occurring for relative toe depth of $h_t/L_p \approx 0.01$ in both cases.

The trend of decreasing relative scour depths with increasing relative depth (for $h_t/L_p > 0.012$) fits with the form of the scour prediction formulae devised in [4] and [9] where measurements were made within this range.

The trend of increasing relative scour depths for increasing relative depth (for $h_t/L_p < 0.012$) fits with the form of the scour prediction formulae devised in [3] where experiments showed increasing relative scour depths for $h_t/L_p \leq 0.015$. Some authors, such as [16] had considered that the variation of scour depth with relative depth in [3] ran contrary to expectations and other scour formulae.

These tests have reproduced the form of the results in [3] within the expected range. This illustrates the fact that scour occurs by different mechanisms in different hydrodynamic regimes. The different scouring mechanisms should not be expected to exhibit the same variation in scour depth with relative water depth. These tests have helped to reconcile the approaches of [3], [4] and [9].

Fig. 7 (top) shows that accretion occurred for the lowest and in one case for the highest relative water depths. Accretion occurred at the toe of the vertical structure for $h_t/L_p \approx 0$ due to swash zone processes. At high relative depths the lack of wave breaking resulted in a scouring pattern dominated by streaming and similar in form to those in [9]. In these cases accretion at the toe of the structure can occur.

Fig. 7 illustrates the differences between maximum and toe scour depths, which are shown over the same ranges and at the same scale to aid comparison. Maximum scour depths are all positive and are always larger than or equal to the toe scour depth. The maximum relative scour depth recorded was $S/H_s = 0.82$.

XI. Variation in Scour Depth with Iribarren Number

The observed dependency of the scour depth on the form of wave breaking on the structure indicates that there might be a relationship between scour depth and Iribarren number [17] (or surf similarity parameter as it is also known) defined in Equation 6 and including the beach slope, $\tan(\alpha)$. 
psi \tan \alpha = \frac{\tan(a)}{\sqrt{H_{si}/L_p}}. \quad (6)

On a uniformly sloping beach without a seawall the breaker type has been categorized as spilling for \( \text{Ir} < 0.5 \) and plunging for \( 0.5 < \text{Ir} < 3.3 \) [18] although there is no abrupt limit from one breaking state to the other for irregular waves. In this case the wave breaking in front of the structure was heavily influenced by the reflections from the structure (with reflection coefficient in excess of 0.8, see Table II). This has resulted in waves plunging onto the seawall for Irribarren numbers less than 0.5. Fig. 8 shows the variation of relative scour depth with Irribarren number for toe scour (top) and maximum scour (bottom).

There is a stronger apparent link between the relative maximum scour depth and Irribarren number than there is between the toe scour depth and the Irribarren number. The simple linear fit given in (7) had the equal lowest mean absolute error of 0.134.

\[
\frac{S_{\text{max}}}{H_{si}} = 1.30 \times \text{Ir} + 0.169. \quad (7)
\]

However, there is a considerable variation in scour depths for similar values of Irribarren number. For example Tests 4, 7, 11 and 12 all have Irribarren numbers between 0.29 and 0.31 but have relative maximum scour depths between 0.16 and 0.82. All four tests have incident significant wave heights between 0.194m and 0.217m, peak periods of 3.24s and were performed with a vertical seawall and an initial beach slope of 1:30. The difference lies in the toe depth, which governs where the seawall is in relation to the position waves start to break and hence which hydrodynamic processes dominated. See section VI for a discussion of these tests and Fig. 2 for their final bed profiles.

XII. VARIATION OF RELATIVE SCOUR DEPTH WITH IRIBARREN NUMBER AND RELATIVE TOE DEPTH

Section X and XI have shown how scour depths vary with relative depth and Irribarren number. Fig. 9 shows relative scour depth plotted against relative toe depth, with the data arranged into three ranges of Irribarren number:

- \( \text{Ir} < 0.08; \)
- \( 0.1 < \text{Ir} < 0.2; \) and
- \( \text{Ir} > 0.2. \)

Fig. 9 shows that for any given relative depth, \( h/L_p \), the greatest scour depths tend to occur for the larger Irribarren numbers. The trend appears more visibly obvious for the maximum scour depth than for the toe scour depth. This opens up the possibility of developing a scour predictor that is a function of relative toe depth and Irribarren number.

XIII. RANGES OF PARAMETERS AND LOGICAL LIMITS TO SCOUR DEPTH AT HIGH AND LOW TOE DEPTHS

The tests performed at HR Wallingford were within the following ranges:

1 \( 0.000 \leq h/L_p \leq 0.073; \)
2 \( 0.059 \leq \text{Ir} \leq 0.430; \)
3 \( 0.00 \leq h/H_{si} \leq 2.08; \)
4 \( 0.006 \leq H_{si}/L_p \leq 0.052; \)

Moreover, not all sections of these ranges were covered equally. Therefore, as with any set of experimental results, any extrapolation outside these limits (and for some cases within these limits) carries a risk. Nevertheless, some limits can be placed on the expected behaviour due to our understanding of the physical processes involved. If the beach extends above the maximum runup limit for a particular seastate, the waves will not reach the seawall so the scour depth at the wall is expected to be zero. Note however, that wave activity seawards of the beach toe may cause scour that may, in time, extend to the seawall toe.

At the other extreme of very deep water the wave orbital velocity will tend towards zero and again no scour would be expected to occur. For relatively deep water and low (or flat) bed slopes accretion also occurs at the seawall for suspended load sediment transport [9].

Figure 8. Variation of relative scour depth with Irribarren number for toe scour (top) and maximum scour (below)
Measurements of toe scour at seawalls in the inter-tidal zone [19, 20, 21] show that scour holes can often form and refill within a single tide, leaving the beach level after the event similar to, or the same as, the beach level before the event. The full extent of such events cannot be determined from beach profiles.

Test 10 modeled a half tidal cycle using bursts of 300 peak wave periods at a series of discrete water levels, starting at 0.05m above the intersection of the vertical seawall and the initial 1:30 bed profile. The water level was then increased in steps of 0.05m to 0.30m before decreasing in steps of 0.05m to a level of -0.10m. Two additional bursts were then added, at levels of -0.08m and -0.09m to see if more infilling of the scour hole would occur.

The same offshore wave conditions were used for all water levels, with a target significant wave height of 0.20m, and spectral peak period of $T_p = 1.87s$. The gains on the wavemaker and the wave absorption system were tuned to the water depth before running each burst of waves to ensure that the wave conditions were as uniform as possible.

The bed level change profiles from Test 10 are shown in Fig. 10, which is split into three sections. Fig. 10a shows the scour profiles from the rising water levels. The scour depth at the wall increased up until 1200 waves (with a water depth at the toe of 0.20m above the initial bed level) then decreased as the water level rose to 0.30m after 1800 waves. This is compatible with the results from Section VI and the observations that the greatest scour depths are achieved when waves break directly onto the seawall. This was observed to occur at a depth of 0.20m than at higher or lower depths. The position of the maximum accretion (within 1.5m of the seawall) moves offshore from the seawall as the water depth and hence wavelength increases.

Fig. 10b shows the scour profiles from the greatest water depth of 0.30m (the last profile shown in Fig. 10a) down to a depth of 0.05m, after 3300 peak wave periods. Here the scour depth at the wall did not increase noticeably as the water level dropped from 0.30m to 0.20m, but it did increase as the water depth dropped from 0.20m to 0.10m. At the same time the location of maximum accretion (within 1.5m of the seawall) moved towards the seawall and decreased in elevation as the water level dropped, until after 3300 waves there was no accretion within 1.5m of the wall and the whole section of the seabed exhibited erosion. Between about 2m and 3m offshore a second area of accretion remained above the initial bed level.

Figure 10c shows the scour profiles from a water depth of 0.05m down to -0.10m then 2 after 2 further bursts of waves at -0.08m and -0.09m. At these low water levels the further offshore mound (between 2m and 3m from the seawall) became gradually washed out and the bed profiles became smoother. The toe scour in front of the seawall also started to fill in by a small amount. The water depths for the final two bursts were chosen so that the offshore mound was just exposed as it seemed to be periodic swash events over this outer mound that contributed most to the infilling.

Figure 10c shows that the nearly complete infilling of scour holes seen in the field was only partially reproduced in the medium scale laboratory tests. Scale effects and the discrete representation of changes in water level as well as the longshore non-uniformity of bed levels are all likely to have contributed to the differences between the results in the laboratory and measurements made in the field.
Figure 11. Bed level changes (from initial bed level) after a series of bursts of 300 waves at different levels to simulate half a tidal cycle.
relative water depth at the structure toe, \( h \). The structure also means that the toe may slide outwards – cause its sudden collapse. A deep scour hole at the toe of failure by sliding.

Any slip surface, thereby increasing the risk of structural failure. A scour hole away from the seawall, leaving a void behind the seawall that may contain fill material to escape under immediate adjacent to the toe of the structure and \( S_{\text{max}} \) the maximum scour depth measured at any point in the test section. Both are of interest in considering the stability of coastal structures. The presence of a deep scour hole at the toe of a structure may allow fill material to escape during a half tidal cycle, as observed in the field. Only partial infilling occurred, probably as a result of scale effects and the discrete changes in water level made during the test.

The relative scour depth was found to depend on the relative water depth at the structure toe, \( h/L_p \) and the Iribarren number (6). It was relatively insensitive to the slope of the seawall, with a 1:2 slope giving similar scour depths to a vertical seawall.

An attempt was made to reproduce the formation and infilling of a scour hole during a half tidal cycle, as observed in the field. Only partial infilling occurred, probably as a result of scale effects and the discrete changes in water level made during the test.

**REFERENCES**


