History of Alderney and Jersey "harbours of refuge" –why did they fail?

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Abstract

During the early 1800s, Britain still feared the 'Napoleonic threat' from the French Navy and their allies. That fear was used by Parliament to justify the construction of various coastal and harbour schemes, activities in which ICE members were actively engaged. The more explicit threat from France abated with the defeat of Bonaparte's armies at Waterloo in 1815, and his death in 1821 on St Helena, but the momentum of some schemes continued, and fears of a French resurgence fuelled proposals for 'harbours of refuge'. In the Channel Islands, the most notable such harbours were those at Braye Bay, Alderney; and St Catherine's, Jersey. Construction of the breakwaters to form both of these harbours started in 1847. The breakwater at St Catherine's was complete in 1856. The Alderney breakwater was complete to its originally intended length by 1871, but was thereafter abandoned to part length.
This paper will discuss the 'official' reasons given for constructing 'harbours of refuge', and will point to some of the real reasons. It will discuss the two Channel Island sites, and the design and construction of the (three) main breakwaters.

Both harbours failed in different ways. The original 'client needs' were significantly modified or their requirements simply disappeared. The paper will outline how and why these harbours / breakwaters failed, whether they were ever viable.

1. Reasons for 'harbours of refuge'

Harbours of refuge were notionally conceived to provide shelter from storms for commercial vessels, including mail packets, fishing and general trade. At the time of the design of these harbours (~1840) such vessels would have been powered primarily by sail. Harbours and trading practices would have adapted to the restrictions that imposed. For instance, it was very difficult for a sailing vessel to leave harbour into a directly onshore wind. This was understood, and allowed for in the commercial sector.

The hidden sub-text of the 'harbours of refuge' debate was however availability of new harbours for the Royal Navy for deterrence, i.e. defence. The further sub-text (less commonly discussed) was the potential use for such harbours for offensive purposes. For the Channel Islands, this would essentially be to stage attacks on (or blockades of) the major French ports, particularly Cherbourg.
Possible sites for 'harbours of refuge' were at: Holyhead, Peterhead, Harwich, Dover, Seaford, Portland, Jersey (St Catherine's) and Alderney (Braye Bay). This paper will discuss the last two.
Both Jersey and Alderney are close to the coast of France, perceived at the time to be a major military threat. So whilst possible harbour developments here might be shrouded with the cloak of 'harbours of refuge', the truth is probably that these two harbours were all along simply military enterprises, and the hand of the Admiralty may be detected pulling strings behind the scenes.

Alderney

The island of Alderney is just to the west of the major French naval port of Cherbourg in an area of high velocity tidal streams, the 'race of Alderney', Figure 1. The western coast of Alderney is exposed directly to Atlantic storms. As a possible base for a harbour of refuge, Alderney is therefore well south of any coastal traffic along the south coast of England. Most trans-Atlantic trade in the 1800s was run from either Bristol or Liverpool, so far to the west of the Channel Islands. Almost no civilian vessels would use a refuge harbour on Alderney. And even if they could, then they would prefer to shelter on the less wave-affected south-east side of the island.

The (probable) truth behind the selection of this site is suggested by its closeness to Cherbourg. At the time, a major threat espoused by naval tacticians was the blockade, where one fleet traps their enemy's fleet within its own harbour, one of the reasons behind such harbours using two entrances, as at Cherbourg and Dover. So a convenient harbour to house an offensive fleet close to their enemy's main harbour was probably highly attractive to the UK Admiralty.

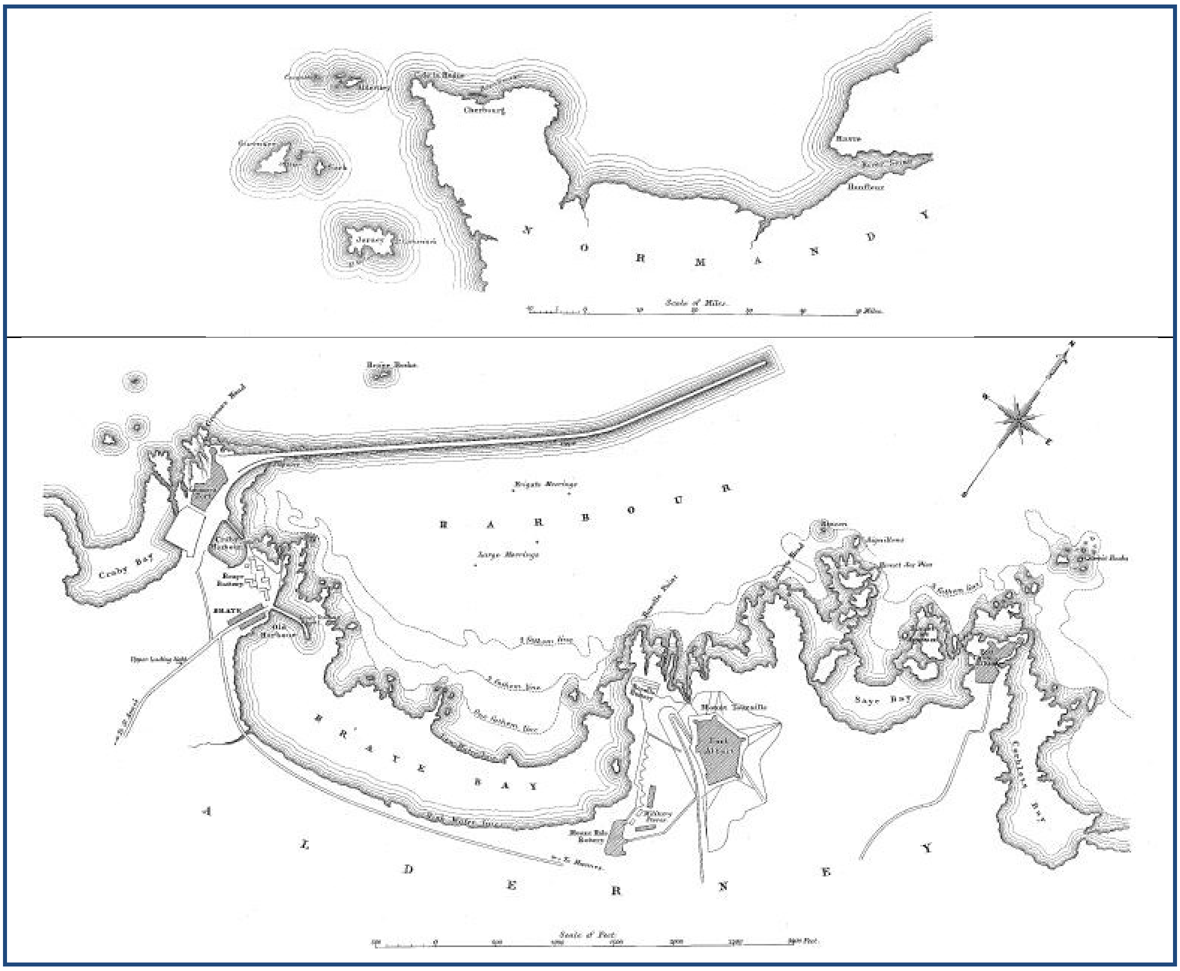

Figure 1: Location of Alderney and Jersey above; below Braye Bay and Alderney Breakwater (from Vernon-Harcourt (1873) courtesy ICE proceedings)

If that was the real reason for siting this new harbour on Alderney, then why put it on the coast most exposed to waves? Again the reason was military. It was to hide any British fleet from the telescopes of the French on the cliffs of the Cherbourg peninsular. But in choosing to locate the harbour on that side, Admiralty planners effectively sealed the ill fate of the harbour, and certainly of the breakwater.

Some of the background to the selection of these sites is discussed in ICE proceedings, and much is presented by Davies (1983), mainly for St Catherine's, but also for Alderney. In searching for government reasons for the selection of these sites however, Davies notes that reasons "… *have not been easy to trace because Admiralty papers on this delicate subject have been … 'weeded"*.

In discussions to the ICE paper on Alderney by Vernon-Harcourt (1873) some 26 years after construction had commenced, Admiral Sir Edward Belcher explained that he had been summoned by Government in August 1842 "*… on secret service …*" to examine (military) defences in the Channel Islands and advise on "*… what guns should be added or withdrawn, and what harbours should be made…*" He was asked to report as early as possible to allow estimates to be laid before Parliament. At Alderney, they found the tidal race across "*… the mouth of the proposed harbour…*" [one assumes here at Braye Bay] "*… would render it utterly impossible for any disabled vessel to get in…*". He had suggested locating the harbour at Longy (on the south-east side of the island). His advice to the Admiralty in September 1842 was that a harbour at Longy would cost £1,500,000.

In 1852 (so five years after construction at Braye Bay had started), Sir Francis Baring summoned him back to the Admiralty to tell him that "*… the former Commission was still in force … ordered to go to Alderney harbour and report upon it*." Further "*… you are not to entertain any of the opinions that you entertained before; you are to examine the place and tell us what has been done, and whether it is worthwhile to expend £600,000 more on the eastern arm*." James Walker (by then the designer of the Alderney breakwater) was also instructed to go "*… in order that he might be there in a gale.*" It appears that both Walker and Belcher advised against the (additional) eastern arm, perhaps convinced that the concomitant concentration of tidal flows across the breakwater heads would scour their foundation mounds. Belcher concluded his contribution to the Vernon-Harcourt (1873) discussion with the barbed comment: "*The present works were certainly a credit to British engineers, and showed what Englishmen could do when they were determined – whether right or wrong*."

In the same discussions, Vernon-Harcourt (1873) noted in an extensive response that the idea of the eastern breakwater had not been abandoned until 1862. Whilst agreeing with Sir John Coode and Colonel Jervois that the eastern arm should be added " *… if the harbour was to be rendered perfect …*" he felt that it was little use as a 'harbour of refuge' being away from the main shipping routes, and it was "*… a bad harbour in easterly gales*." He disagreed with Sir Edward Belcher on the 'rapid scouring' fear "*… as the harbour area was not large and the rise of tide at Alderney was not peculiarly great*", but then he was probably not taking full account of current velocities along the breakwater.

Jersey

Again, there are two issues that affect the potential utility of any 'harbour of refuge' on Jersey: whether Jersey is a useful location at all; and if so, where on Jersey might one be constructed? Again the processes of decision-making in Parliament, the civil service, and the Admiralty have been obscured by the 'weeding' of papers referred to by Davies (1983), so we must work hard to reconstruct the possible reasons. Davies mainly gives the reasons for 'why not'!

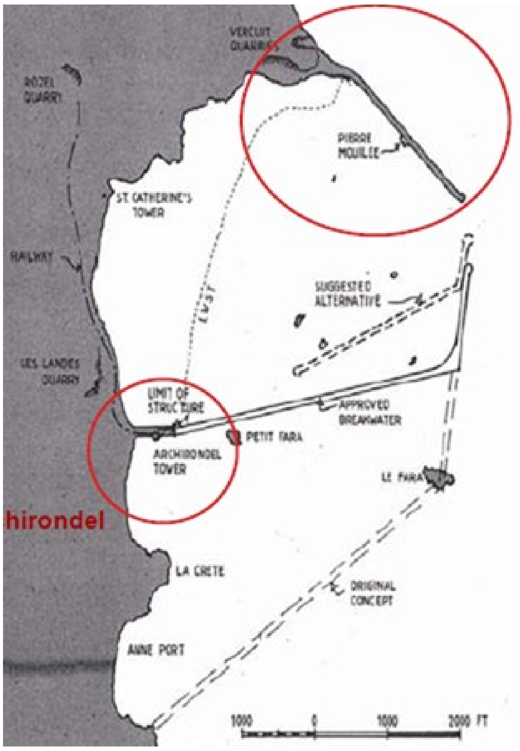

Figure 2: St Catherine's, from Davies (1983)

Before discussing 'why', it may be helpful to identify what was proposed. The plan shown by Davies (Figure 2) starts with two breakwaters, both of which were indeed started in 1847: St Catherine's to the north; and Archirondel to the south. The St Catherine's breakwater exists to this day (Figure 3), indeed Hold (2013) recently reported on its refurbishment. The Archirondel breakwater was planned to be 2.5 times longer, protecting the harbour from southerly and south-east waves, and inter alia from the northerly running near-coast tidal currents. But in July 1849 Walker instructed the contractor Jackson & Bean to stop work, notionally to divert effort to the completion of the northern breakwater, but probably due to the appreciation of a shortage of depth, perhaps as the putative harbour started to silt up with the northern (St Catherine's) breakwater trapping the sediment laden northerly drift. A mere stub of the Archirondel breakwater exists today, Figures 2 and 3, probably in somewhat similar state to that in which it was abandoned.

It is difficult, indeed pretty much impossible, to disagree with Davies that siting any true harbour of refuge on Jersey made no sense at all. This is an island of 12m tides, surrounded by inter-tidal rocks and islets. It is close to, but separate from the coast of France to which it is nearly 'joined' by a submerged line of rocks that run out east-south-east from the Jersey coast to near Coutances on the French coast. These rocks, together with the substantial tidal flows between Jersey and France, significantly limit any trading vessel traffic along the east side of Jersey, so most vessels from UK pass to the west. In any event, very little traffic originating from the UK for further destinations will pass anywhere near Jersey, unless trading direct with the Chanel Islands, in which case it has little use for a 'harbour of refuge' per se. Again, almost no civilian vessels would use a refuge on Jersey.

So what about military use, even if not declared as such? And if so, where? Here Davies (1983) rehearses the arguments at some length, with much of the convoluted discussions. In 1831, William Symonds (later Sir) wrote to the Lieutenant-Governor (of Jersey) assessing naval activities on the French coastline, and appraised options for harbours around Jersey. In this, Symonds favoured Bouley Bay on the north coast, although this was opposed by (Admiral, Rtd) Martin White (naturalised Jerseyman and navy surveyor of some note) who "*unmistakably showed up the defects*" of that option.

During the early 1840s, the issue of a new harbour on Jersey was complicated by the involvement of Sir William Napier, Lieutenant-Governor of Guernsey, who appears to have been inveigled by Whitehall "*to prepare a military appraisal of the Channel Islands as a whole*", for which "*he personally inspected Jersey, Guernsey, Alderney, Sark and Jethou*". Sir William was not impressed by the civilian administrations of either Jersey or Guernsey, and "*crossed swords with everybody who did not agree with his point of view, whether they be military or civil* ". The UK government then set up a Commission to revisit Sir William's work. Commission members included Admiral Belcher, Colonel Cardew, Lieutenant-Colonel Colquhoun, supported by James Walker, Captain Sheringham (surveyor), some of whom were later involved in the Harbours Commission of 1844.

Figure 3: St Catherine's, top; and remains of Archirondel, bottom (author, 2014)

But by 1842, Government was minded to act. There were competing claims for Noirmont Point on the south-west coast of Jersey, or Bouley Bay towards the north-east corner, or none at all. For reasons that are still opaque, and not supported by any of the main protagonists, the government opted to construct the new harbour at St Catherine's Bay on the north-east coast of Jersey. Davies comments that the reasons to go against advice by Martin White (the most experienced local surveyor) for Bouley Bay cannot be found " *because Admiralty papers … have been … weeded*".

Davies notes that the Harbours Committee of 1844, set up by the Lords Commissioners of the Treasury, did not mention the Channel Islands, yet in only three years, both "*the St Catherine's and Alderney projects had been proposed, authorised and commenced. No sound reason can be found for such a hasty decision, and this aspect must remain a mystery."* The 'haste' is illustrated by the Act dated being 2 April 1844 and the report being submitted to their Lordships on 7 August 1844.

But what were those main objections? Firstly, even if a harbour could be maintained, its utility would have been severely limited by the tidal conditions for which it could be accessed, simply when there would be sailing space (of appropriate depth and with manageable currents) between Jersey and France. The second issue was the threat of siltation, particularly of sand driven by waves and currents on the north-ward running tidal flows, therefore likely to enter the proposed harbour under each flood tide, and then tending to deposit over slack water. If that was not potentially bad enough for a completed harbour, the early cancellation of the southern Archirondel breakwater (Figures 2 and 3) increased the opportunity for the nearshore (sediment laden) current to be trapped by the northern St

Catherine's breakwater, and this (probably) significantly increased rates of siltation. Indeed it was the apparently rapid reduction of depth within the putative harbour area that was cited in the 1849 Admiralty decision to halt and then abandon any further progress on the Archirondel breakwater.

2. The generality of harbours of refuge

In considering the options above, it is worth noting that developments of steamships were in their infancy in 1830-40 (Barnes, 2014), but that over the following years requirements for harbours (particularly naval harbours) were significantly altered by the changing forms of propulsion, particularly the reduced mooring and swinging space required, and the ability to depart under adverse wind directions. This was potentially of significant benefit to the French ports at St Malo and Granville (perhaps also at Cherbourg) where the new steamships would more easily depart under prevailing Westerly winds than would sailing vessels.

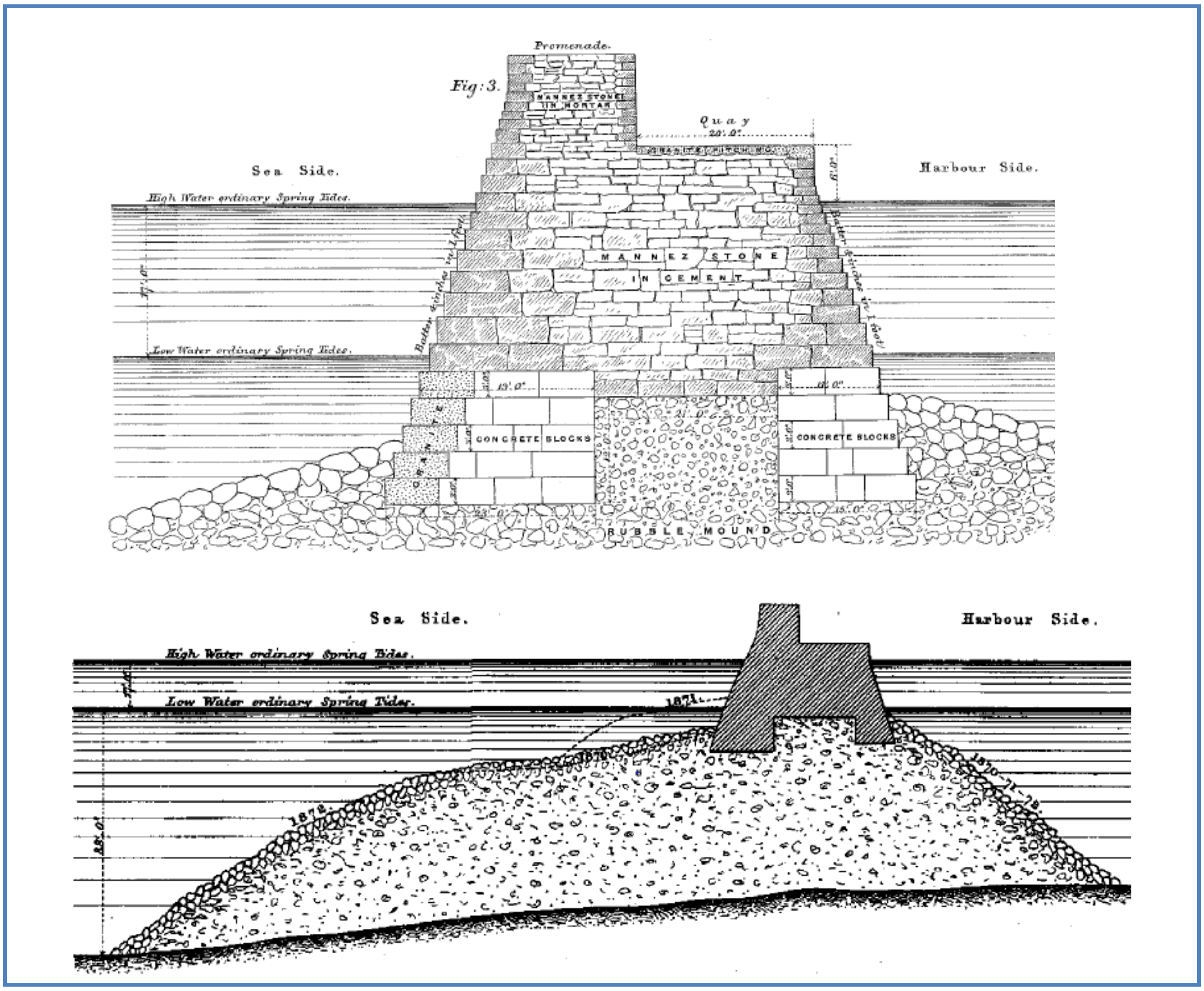

Figure 4: Sections of mound and breakwater wall, Alderney. Note extended mound from addition of stone dumped down the seaward face of the wall.(Source: Vernon Harcourt, 1873)

The often heated discussions at ICE on 'harbours of refuge' may have been fuelled in some part by struggles for prominence, and the apparent proximity of a large pot of money. This might be illustrated by a discussion following presentation of a paper on Blyth by Scott (1858). Mr Bidder (ICE Vice President) discussed the generality of Government supervision of the 'harbours of refuge', primarily Holyhead, Portland, Dover, and Alderney. Bidder had found it necessary to examine "*the formidable and not very lively documents, the Parliamentary Blue Books… which confirmed his own previous observations …these great works were being executed without any efficient responsible supervision or control* ", asserting further that *"… the Government itself had been kept utterly in the dark… The time had now arrived when these matters should be brought before the bar of public opinion … the Institution of Civil Engineers appeared to be the most fitting arena for the discussion of the question*." Bidder referred to several Reports of the Committee on Harbours of Refuge from 1845, noting that they could not agree on the preferred form of breakwater, "*…chiefly arisen from the Committee not having arrived at a clear understanding of the terms used, and of the basis of the various arguments employed*." He continued [somewhat acidly] " *… facts derived from the Blue Books … appeared to contain everything except the specific information sought for*."

Considering Alderney, the section "*appeared to be of a disadvantageous form … the effect of the waves upon this wall must be very prejudicial … and greater than upon any other form which could be devised.*" Bidder continued in an attack on James Walker (past President of ICE, and designer of both breakwaters at Alderney and St Catherine's) who had signed the report of 1845 stating that the costs of a vertical wall or rubble mound "*would be nearly identical*". Yet the vertical pier at Dover was costing £415/ft, whilst the rubble mounds at Portland less than half that. Of four works recommended, three had been commenced, and two "*had been intrusted* (sic) *to Mr James Walker, himself one of the Commissioners*". He continued "*… it seemed that the Government authorised works …without any idea being given of the cost of such works, or of the time that would be occupied in their construction, or even of the mode in which they were to be executed*."

Bidder then turned to the harbours on Alderney and Jersey, the former being "*nearly valueless*" and that at St Catherine's offering "*scarcely shelter for a few fishing boats*". In conclusion, Bidder criticised [in fairly immoderate language] the shortage of independent members in the Commissions, the prevalence of "*foregone conclusions*" and "*hocus pocus*" in decision-making. He called for "*the attention of some independent Member of the House of Commons … pertinaciously attacking and exposing the present objectionable system …*"

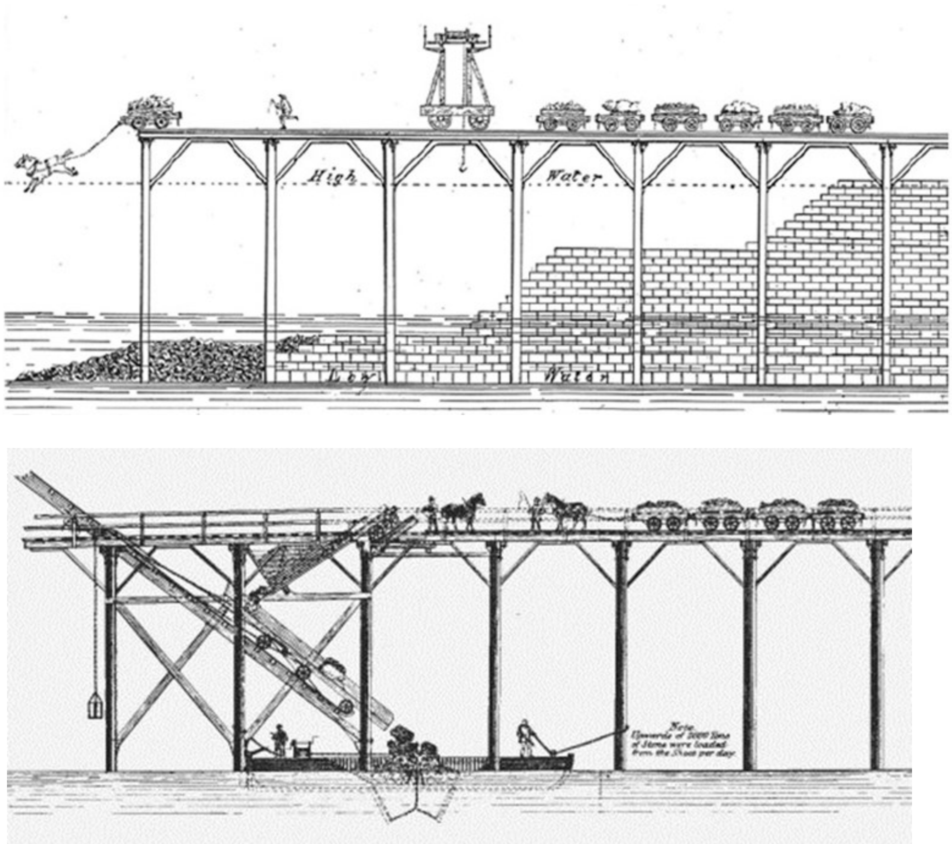

Figure 5: Construction trestles, St Catherine's (Davies (1983), top; the rock chute at Alderney (Hold 2009), bottom. [The frightened horse landed in the water and was not severely injured.]

3. Breakwater design and construction

The Walker designs for both of these breakwaters were essentially the same, although that at Alderney was progressively modified as each iteration suffered further damage. The initial breakwater design at both sites used a mound of quarried stone to low water, surmounted by blockwork walls with rubble infill. Most of the stone for the mound and walls was quarried locally, from the Mannez quarry on Alderney, or Verclut on Jersey, although both required imported granite facings to reduce erosion.

Shortly into construction, the design at Alderney was revised. The mound level was reduced to -3.5m to -4mLW to try to improve stability of the foundation stones. Those foundation stones, until then simply placed tightly, were now laid using cement mortar (commercial production of Ordinary Portland Cement had recently started, and helmet divers became available). The batter of the wall itself was steepened to give a greater 'pinching force' on the lower blocks. This construction, continued to a length of 823m by 1856. The design was then revised again and construction of the outer section was completed in 1864, giving a total length of 1430m.

At both sites, the main construction was from above, supported on timber staging with little steam power to assist. At Alderney, an innovative rock chute was devised to get rock into the barges without simply punching a hole through the bottom of the barge! Rock slid down the chute was slowed by the change of direction at part-height, see Figure 5. It appears that mound rock at St Catherine's was simply tipped from the staging. Here the greater tidal range, and lower wave exposure, made placement of the wall blocks in the 'dry' far easier.

In considering the apparent similarities of the starting design, and the later changes at Alderney, it may be instructive to review the rapid changes in hydro-dynamic understanding, and of materials and equipment available (e.g. Figure 5). It may also be useful to summarise the great differences between the sites, and particularly in the way that waves attack these breakwaters.

4. The glories of hindsight

Wave loadings

At the time of the design of these breakwaters, ~1845-47, breakwater design was essentially by trial and error with no calculation of loads or resistance. Designs advanced by experience. Two comments from the time give an indication of the problem. Scott Russell J.(1847) remarked: "*Perhaps it may be considered rather hard by the young engineer, that he should be left to be guided entirely by circumstances, without the aid of any one general principle for his assistance.*", Then in discussing his innovative wave dynamometer, Stevenson (1849) remarks:"*… the engineer has always a difficulty in estimating the force of the waves with which he has to contend….. The information … derived from local informants … is not satisfactory*." Those uncertainties were substantially compounded by very significant general misunderstandings on wave behaviour over submerged mounds, although not for want of trying many different descriptions. Here the two text books by Vernon-Harcourt (1885), but more particularly that by Shields (1895), might have been helpful, had they been available to Walker in 1845-47. Even without the assistance of design formulae and guidance on near-structure wave transformations, it is still a little surprising to modern eyes that the designs were so similar when the exposure was so different.

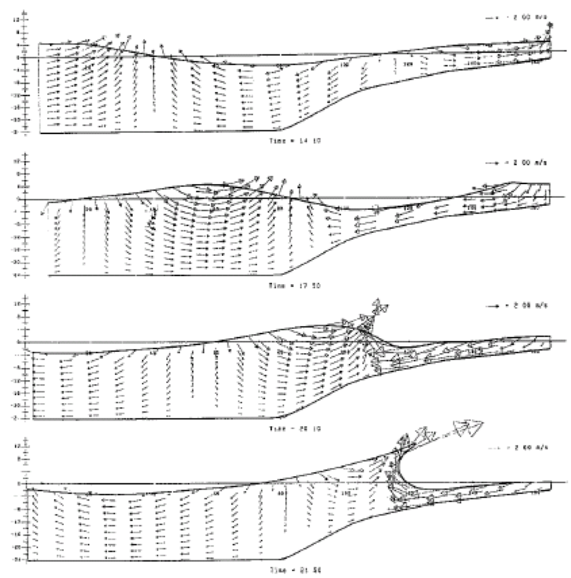

Figure 6: Numerical modelling of wave action over the mound at Alderney, from Allsop *et al* (1991)

The site at St Catherine's on the lee side of Jersey is essentially sheltered from all major storm waves. Waves from the Atlantic are substantially reduced by refraction and diffraction along the north coast of Jersey. When they reach St Catherine's the remaining waves are strongly oblique to the breakwater. The only direct attack on this breakwater will be by waves from north and east which are strongly fetch-limited. The tidal range at Jersey at ~ 12m is one of the greatest in the world (a few sites reach ~14m), but the general tidal currents are not focussed here, except in local flows around the roundhead. So this breakwater is very lightly attacked, as evidenced by the significant lack of damage or demand for repair until very recently.

The conditions at Alderney could hardly be more different. The tidal range is less at 5.2m, but tidal currents may exceed 7-8 knots in the Race of Alderney. Numerical modelling of waves and currents discussed by Allsop *et al* (1991) show that waves are refracted by these currents in somewhat surprising fashion. It is often expected that tidal currents are greatest at mid-tide level, with slack water at high and low tide levels. At Alderney the contrary is true with tidal velocities being greatest around high and low water. Those high currents reduce wave heights at the breakwater at high and low water, but no wave-current refraction applies at mid-tide so wave attack is greatest. Modelling in 1989 (see Allsop *et al*, 1991) gave a 1:50 year condition of H_s=11.0m offshore reducing to H_s=8.0 to 8.5m at the breakwater. Sadly the combination of direct wave attack at mid-tide, and the attendant depths over the submerged mound, have the malign effect of shoaling waves to break impulsively onto the breakwater wall, Figure 6.

The effect of this on wave forces on the wall calculated using the simplified Goda – Takahashi method in Figure 7 show greatest forces around mid-tide rather than at high tide. This method does not try to calculate impulsive loads *per se* (see Allsop, 2000). For that, the more complicated approaches of Cuomo *et al* (2010, 2011) would be needed, but the simplified calculation illustrated here suffices to illustrate that forces are not maximised by considering only the highest water levels.

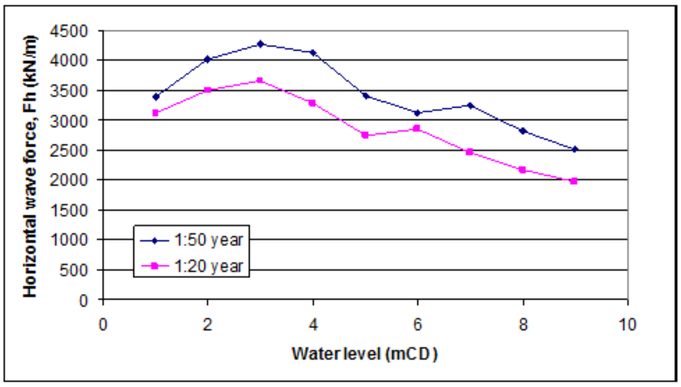

Figure 7: Simplified Goda / Takahashi forces on the Alderney breakwater wall

The debate on wave behaviour was discussed by Shield (1899) who reminds his reader of "*…one or two leading points … generally accepted as the theory of waves.*", discussing the change from circular wave orbits to ellipses as waves move into shallow water. He notes that waves "*break on entering water of a depth which but little exceeds their height…*" [implying that the effects of steep bed slopes, and (perhaps) wave period on wave breaking limits were little appreciated]. The following comment *"… swell waves however … are often transformed into waves of a dangerous character*" whilst being somewhat oblique, does illustrate a growing appreciation of these effects. Shield then uses work by Airy (1848) to derive relative particle displacements for various depths below the water surface, concluding that, for all depths in which it is practical to construct breakwaters, storm waves will (mostly) have transformed to "*waves of translation* ". In discussing wave action at a vertical quay with an approaching bed slope of 1:10, Shield noted "*As the tide recedes, however, they are quickly transformed into angry waves of translation by being tripped up by the foreshore…".* He then draws the similarity with Alderney, noting that the returning wave often causes damage to the foundation, and that high parapets "*greatly intensify this action… and are objectionable*". He notes that rubble may be washed away at the outer end of a breakwater down to depths >12m. At Alderney, with a bed depth of -14mLW at 300m from the root, the mound at -1mLW was not stable even at a slope of 1:6.5, the foundation being withdrawn leading to breaching.

Construction practicalities

But not only did the lack of clear understanding on wave forces severely hamper the design, but key technologies that would greatly assist construction at the end of the century were yet to be developed. Ordinary Portland Cement (OPC) had been patended by Aspedin in 1823, but was not available in commercial quantities until 1840-50. Similarly, construction at Alderney started without use of divers.

Cement mortar (initially Medina, later OPC) and helmet divers were however both included in the design revisions. In discussion to Vernon-Harcourt (1873) John Jackson (the contractors' agent 1857-1866) described using helmet divers to excavate holes to receive support piles. Six divers operated at any one time, four on the sea-side, and two on the harbour side, working in four hour shifts, three shifts per day. Jackson discussed the operation of delivering blocks to the divers, and then to the masons once the blockwork emerged above LW. Medina cement mortar brought fresh from the Isle of Wight so that its setting was not impaired was used in 1 part cement to 2 parts sand to bed the blocks. In arguing for its continuing maintenance, indeed completion, he then claimed that *"… with a small fleet in Alderney and Portland the Channel would be completely blockaded …"* noting that Cherbourg was at the time "… *the finest artificial harbour in the world…*". [So much for the non-military pretence of the 'harbours of refuge'.]

5. How did they fail?

Alderney

Even early during construction, the Alderney breakwater was damaged on multiple occasions, with the wall sometimes being breached completely. That led to various changes of section design, including incorporation of cement mortar in placing the foundation blocks. Even so, damage continued, although Vernon-Harcourt (1873) claims that most had been at points where the mound crossed / intercepted rock outcrops and that other instances of damage were relatively minor. A storm in January 1865 however forced two breaches, both completely through the superstructure, over widths of 15m and 40m. Another breach occurred in January 1866, a smaller one in February 1867, and another 18m wide in January 1868. There were further breaches in December 1868, and fresh ones in February and March 1869. In early January 1870, there were two breaches along the outer part, and five other locations of damage. In consequence of the repeated damage, Sir John Hawkshaw (President ICE) and Col. Sir Andrew Clarke were requested by the Board of Trade (who had reluctantly inherited the harbour from the Admiralty) *"to visit Alderney and to report on the best measures for securing permanently*", either the whole (1740m) or the inner portion (870m) of the breakwater. They visited in May 1870, noted the instability of the mound and suggested the removal of the promenade wall, and deposition of a large additional foreshore of rubble or concrete blocks. The government did not however consider that the costs were merited, so no significant recommendations were implemented.

The wall had been (partially) protected by stone dumped to maintain the foreshore level. About 300,000 tons were tipped between 1864 and September 1871, after which the *de facto* decision was made to abandon the outer length. From 1873, repair and maintenance work covered only the inner length of 870m. The outer portion was abandoned to the sea and the wall quickly collapsed, leaving a mound crest about 4m below low water. For the shortened section, approximately 20,000 tons of stone were dumped annually, and further work was still required to repair breaches in the superstructure. Dumping of rock ceased in 1964.

Waves at Alderney are frequently severe. Depths off the breakwater generally exceed 15 - 20m. Atlantic storms reach the breakwater with little reduction, with the 1 in 50 year storm condition of H_s=11.0m offshore corresponding to H_s=8.0 to 8.5m at the breakwater. The severity of wave impact on the wall is then increased by waves shoaling over the mound, causing waves to break impulsively onto the wall. Storms at Alderney usually persist for many hours, so the breakwater is exposed to the full range of possible wave and water level combinations, particularly those which allow waves to break directly against it.

Responsibility for the maintenance of Alderney breakwater was transferred to the States of Guernsey in 1987. Maintenance costs for years up to 1990 were estimated at around £500,000 per annum, excluding the costs of storm damage. That damage takes two main forms. Direct wave impact on the wall shakes the breakwater, and cracks mortar joints. The impact pressures force water into the joints, and into voids behind. Loose rock from the mound is thrown against the wall, abrading the wall by a depth greater than 1m. Over time, the typical size of rubble on the mound has reduced, and the process has generated considerable quantities of sand.

Up to 1990, a team of 8 men repointed the face of the wall above mid-tide level, fill cracks and replace damaged masonry each summer. A team of 6 civil engineering divers carried out repair work at the toe, working both below and above water. During 1989/90, storms battered the breakwater for six weeks. At its peak on 25/26 January 1990, the storm had a return period of about 1 in 25 years, with offshore conditions of H_s=10 to 10.5m. During the next six days the storm subsided slowly, then rose again to H_s > 7m. On 11 and 12 February, storm conditions again exceeded H_s = 9m. This continuous pounding cracked the masonry facing, and a large cavity was formed in the wall. Finally it was breached by an explosive failure clearly audible in and around Braye. Other sections of the structure also suffered damage.

An emergency procedure had been formulated, and permanent repair work was underway within 10 days. The cost of repair work occasioned by these storms was estimated in 1990 at £1.1 million. Studies by Coode & Partners and HR Wallingford explored a number of possible solutions, see Allsop *et al* (1991). Later work on alternative approaches to protecting this breakwater will be described at this conference by Jensen (2017).

Jersey
At St Catherine's, the failure of the harbour was simply one of utility, compounded by the lack of depth, the inherent failings of the location, and by disinterest by the States of Jersey, and the Admiralty. The breakwater itself has suffered very little damage, most being confined to the outer end, described in a previous conference by Hold (2009). The rapid siltation of the harbour area was accelerated by constructing the breakwaters in the wrong sequence, capturing the sediment-laden northerly current by St Catherine's breakwater, rather than deflecting it by extending the Archirondel breakwater. No records exist of the changes of depth, but they must have been sufficient to cause doubts on the wisdom of continuing construction within the first two years of construction.

Acknowledgements
The author is grateful to University of Edinburgh for facilitating this PhD research, of which this forms a part. Support and guidance from Professors Tom Bruce and David Ingram (Edinburgh) and Trevor Whittaker (Queens Belfast) are gratefully acknowledged, together with advice from Professor Roland Paxton at Heriot-Watt University. The book by the late William Davies (1983) has been essential in the compilation of this paper, as has work by HR Wallingford for the States of Guernsey around 1990, and the Minutes of the ICE, particularly the extensive discussions to various papers in the mid to late 1800s. Support and advice from librarians at Edinburgh and ICE is most gratefully acknowledged, as is the generous bequest made by the late Gerald Marshall to fund digitising ICE Proceedings.

References
Airey, G.B. (1845) *Tides and waves*, Encyclopedia Metropolitana.

Allsop N.W.H. (2009) *Historical Experience of Vertical Breakwaters in the United Kingdom*, Proc ICE Conf. on Coasts, Marine Structures & Breakwaters, Edinburgh, September 2009, ISBN 978-0-7277-4129-5, publn. Thomas Telford, London.

Allsop N.W.H. (2010) *Historical Experience of Vertical Breakwaters*, Presentation slides from the Vernon-Harcourt lecture, 22 February 2010, ICE (http://events.ice.org.uk/vernonharcourt)

Allsop N.W.H. & Bray R.N. (1994) *Vertical breakwaters in the United Kingdom: historical and recent experience*, Proc. Workshop on Wave Barriers in Deep Waters, pp76-100, PHRI, Yokosuka, Japan.

Allsop NWH, Briggs M.G., Denziloe T., & Skinner A.E. (1991) *Alderney breakwater: the quest for a 'final' solution*, Proc. Conf. Coastal Structures and Breakwaters, pp 303-320, ISBN 0-7277-1672-7, publn Thomas Telford, London.

Allsop N.W.H. & Shih R. W-K. (1990) *Alderney Breakwater: Phase 3 hydraulic Model Studies*, Report EX 2231, December 1990, HR Wallingford.

Barnes J (2014) *Oceans of power – 125 years of marine engineering milestones*, publn. IMarEST, Aldgate House, London, ISBN 978-0-9565600-8-7

Bishop RW (1950) *Maintenance of some rubble breakwaters: Alderney breakwater*, ICE Maritime & Waterways, Paper 16, Session 1950-51, pp 16-35 & 36-53.

Cuomo, G., Allsop, N.W.H., Bruce, T. & Pearson, J. (2010) *Breaking wave loads at vertical sea walls & breakwaters*, Coastal Engineering, Vol. 57, Issue 4, April 2010, pp424-439, Elsevier Scientific.

Cuomo G., Piscopia R., & Allsop N.W.H. (2011) *Evaluation of wave impact loads on caisson breakwaters based on joint probability of impact maxima and rise times*, Coastal Engineering, Vol. 58, January 2011, pp 9-27, Elsevier Scientific.

Davies, W (1983) *The harbour that failed*, ISBN 0 946346 03 8, The Ampersand Press, Alderney

Hampshire S, Turner R, Hancock D & Guthrie G (2013) *In a Heugh – the strategic influence of a breakwater*, Proc ICE Breakwaters Conference, ICE Publishing, ISBN978-0-7277-5975-7, pp188-198.

Hold S (2009) *Repair of St Catherine's Breakwater roundhead*, Proc. ICE Conference Coasts, Marine Structures and Breakwaters, Edinburgh, ISBN 978-0-7277-4129-5, Thomas Telford, London.

Jensen OJ (2017) *Alderney Breakwater, a developed rehabilitation solution*, Abstract accepted for ICE Breakwaters conference.

Scott M (1858) *Description of a Breakwater at the Port of Blyth; and of Improvements in Breakwaters applicable to Harbours of Refuge*, ICE proceedings, Paper 991, pp72-90, and discussion pp91-161

Scott Russell J.(1847) *On the practical forms of breakwaters, sea walls and other engineering works exposed to the action of waves*, Proc. ICE, Vol VI, pp135-148.

Shield W (1895) *Principles and practice of harbour construction*, Longmans, Green & Co., London

Shield W (1899) *Notes on the Effects of Waves on Breakwaters in Different Depths of Water*, Proc. ICE, Vol. CXXXVIII, pp 398-404

Stevenson T. (1849) *Account of experiments upon the force of the waves of the Atlantic and German oceans*, Proc. ICE, pp23-32 (reported by David Stevenson)

Townshend, B O'D (1898) *Repairs at Alderney Breakwater*, Paper 3006, Proc. ICE, London

Vernon-Harcourt LF (1873) *Account of the construction and maintenance of the harbour at Braye bay, Alderney*, Paper 1347, Institution of Civil Engineers, London.

Vernon-Harcourt LF (1885) *Harbours and docks: their physical features, history, construction equipment, and maintenance*, Clarendon Press Series, Oxford University Press, Oxford.

Wilson, M F-G (1919) *Admiralty Harbour, Dover*, Proc ICE, paper 4274, meeting of 18 November 1919, pages 31-112.

Exploring structural stability of old blockwork breakwaters

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Abstract

Many old coastal harbours in the UK are protected by blockwork breakwaters, but the original streams of income for maintaining or refurbishing these structures have largely diminished. Foundation settlement, loss of mortar, missing blocks, and voiding behind walls or under decks all tend to destabilise these breakwaters, and have in some cases led to localised or complete failures. Those who manage these assets may benefit from an answer to the question: what happens if they fall down? This paper presents results from an exploratory study of these structures' strength, stability, collapse mechanisms, and collapsed form. Construction details and dimensions are extracted from historical documents and an analytical spreadsheet model is used to guide the design of physical model tests in the flume. These tests show sensible collapse mechanisms, despite challenges of scale and model effects. Profile measurements are plotted over the test series, showing the progression of front and rear wall failure, and the collapsed crest level. The companion paper by Allsop *et al* (2017) presents and discusses the residual wave protection offered by these failed breakwaters.

Introduction

The 17th and 18th century UK saw rapid expansion in construction of small harbours, and many blockwork breakwaters were built to shelter them from waves. These structures were commonly formed by rubble mounds built to low-water, surmounted by vertical or battered walls of dressed stone blocks (later concrete) with random rubble core between. The core, or hearting, of these structures is poorly documented, probably quarry-run interspersed with broken blocks and chiselled scraps of the wall-stones. Example cross-sections from St Catherine's and Whitehaven are shown in Figure 1. Many of these breakwaters are now dilapidated, yet some remain the primary sea defences for their harbours. Of concern to both those who manage the structures and those who benefit from their shelter are the following questions. If these structures were to fail, to what height might they be reduced? How much wave protection would they provide in their collapsed form?

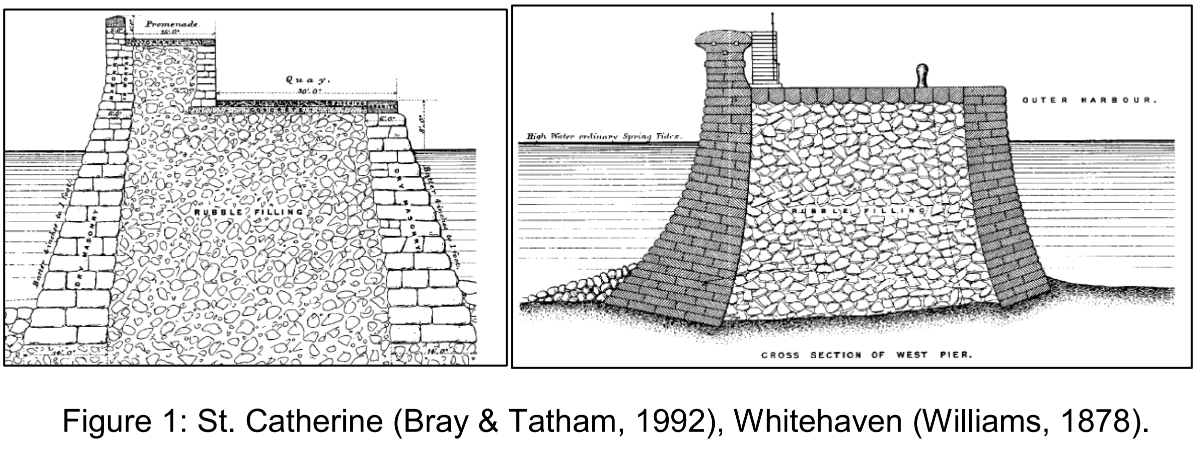

Figure 1: St. Catherine (Bray & Tatham, 1992), Whitehaven (Williams, 1878).

These questions were investigated in simplified 2D hydraulic model tests, conducted to explore wave transmission across damaged and collapsed breakwaters for a set of example structures. The hydraulics of the study are addressed in the sister paper by Allsop *et al* (2017). This paper discusses the study's structural design: scaling the test structures to represent a typical blockwork breakwater and to reproduce a sufficiently realistic collapse process. This was achieved by:

a) Review of historical documents for structural dimensions;
b) An analytical wall stability model;
c) Test-builds to ensure that the design was representative, but would collapse within the test facility;
d) Examination of structural failure under wave action; and
e) Evaluation of 'model effects' and their influence on the results.

Example breakwaters

The initial model design study reviewed breakwaters at Whitehaven, Blyth, Kilrush, Alderney / St. Catherine's, Peterhead, and Hartlepool. These structures are mostly vertical (or slightly battered) blockwork walls with random rubble infill; Peterhead is the exception, being 100% blocks. Historical records contain few construction details, but nominal dimensions were extracted from photos, site visits, and old proceedings of the I.C.E. These dimensions are summarised in Table 1. It is noted that core grading is poorly (or not) documented, but it is expected that the fill will be relatively evenly graded with an upper-limit diameter of a typical wall block.

It is probable that the strength of these breakwaters has relied primarily on the self-weight (and bonding) of the blocks in the walls, and on the stability of the foundation mound. Contributions of any mortar used in construction to overall stability will often be small, particularly where there has been little recurrent maintenance, and the mortar has degraded or been washed away over the years. Self-weight and friction between blocks will therefore govern stability. The distribution of gravity forces throughout the blocks will however be non-uniform. This irregularity of load transfer is an inherent feature of any blockwork, more so for mortar-less masonry, directly related to the precision (or otherwise) of block-cutting and laying tolerances. Additionally, abrasion of blocks and local settlement of the mound will redistribute the forces which clamp the blocks in place.

Table 1: Dimensions of typical blockwork breakwaters

Breakwater	Section			Lower blocks			Upper blocks		
	Height [m]	Width [m]	Block : Fill Volume [%]	Height [m]	Width [m]	Length [m]	Height [m]	Width [m]	Length [m]
St. Catherine's	18.0	12.0	28 : 72	0.8	1.0	1.5	0.4	0.6	0.8
Kilrush	8.9	14.4	36 : 64	-	-	-	0.45	0.7	1.0
Whitehaven	13.4	19.2	31: 69	2.2	-	3.3	-	-	-
Blyth	12.7	20.3	29 : 71	1.7	-	2.5	1.7	-	3.4
Heugh	4.0	10.0	-	-	-	-	0.55	0.5	2.1
Peterhead	15.9	14.0	100 : 0	2.0	2.3	3.9	2.0	2.3	4.1

Key structural failure modes are summarised below, given by Allsop (2009) which follows on from guidance in Bray & Tatham's *Old Waterfront Walls* (1992) and British Standard *6349-2 §7.3.1* (2010):

a) sliding or overturning of a breakwater section as a single entity;
b) global geotechnical failure of the foundation, destabilising the wall;
c) removal of blocks from the wall causing discontinuity, thence structural instability; and
d) local geotechnical failure of the mound, destabilising the wall and/or releasing fill.

Sliding or overturning as a single entity (a) and global geotechnical failure (b) have been relatively uncommon in the UK in recent years, but may have been significant earlier, whereas removal of blocks from the wall (c) and local geotechnical failure (d) are more frequent. The main failure mechanisms are however all governed by the self-weight of the blocks, friction between blocks, lateral earth pressure from the core, and hydraulic pressures. Waves striking the wall raise the phreatic surface within the core, and at different phases of the wave action will apply direct forces to the wall, positive or negative. As the waves recede, there is a temporary hydraulic gradient across the wall acting seaward. This increase in seaward pressure can cause the entire wall, or just a section, to collapse.

Test structure design

The example, or 'prototype', structures must be modelled accurately at flume-scale so that both their stability and collapse can be reproduced realistically. This requires scaling of the blocks, core, and mound. A geometric scale of 1:30 was chosen to give a test section which could fit in the flume and also be collapsed by test conditions within the range of the available equipment. Block sizes were set to 140mm long, 70mm wide, and 30mm high with a density of 2320 kg/m^3. Scaled up to prototype values, these represent slightly denser blocks (2400 kg/m3 correcting for the density of the flume's fresh water versus seawater) 4.2m long, 2.1m wide, and 0.9m high. These dimensions correspond well with Peterhead and Blyth, but are slightly large compared to the other example structures. The model core material was widely graded with a nominal maximum diameter D_{max} of approximately 50mm. This was done to ensure that the maximum rock size was smaller than a wall block. Recalling that a typical core will be formed by discarded wall stones, chippings, and natural tout-venant rubble core, the natural upper-limit is indeed the wall-block size. This maximum size therefore corresponds to prototype D_{max} of approximately 1.5m, which is smaller than Peterhead and Blyth wall blocks. The minimum grain size D_{min} was 5mm (0.15m prototype).

Parameter	*Unit*	*Magnitude*
Number of blocks	No.	12
Block length	m	4.2
Block width	m	2.1
Block height	m	0.9
Still water level	m	8.0
Wave height	m	0.0
Block density	kg/m^3	2320
Fill density	kg/m^3	2670
Block friction coefficient	-	0.7
Fill porosity	-	0.35
Friction angle	°	55
Unit Weight of Water	kN/m^3	9.81

Figure 2: Example input and output from analytical model. Blue represents water level. Fill (not shown) is set to crest of top block. Red cell indicates instability.

The basic components being scaled, the next step was to design test structure height, width, and block configuration. The 2D test structure required a cross-section typical of the prototype structures to ensure realistic collapse mechanisms, but also needed to be constructible and stable within the confines of the flume. An analytical spreadsheet model and test builds were used together to investigate wall failure and guide design. The analytical model inputs block, fill, and water level characteristics to assess the stability of an arbitrary section of the wall, idealised as a single column. It assesses: a) self-weight of and friction between the blocks; b) hydrostatic and lateral earth pressures, including buoyancy effects; and c) sliding and overturning equilibrium of each block and set of blocks. This method of stability assessment is consistent with guidance in *Old Waterfront Walls* (1992) and British Standard *6349-2* (2010). Example input and output are shown in Figure 2. The figure shows a cross section of the simplified wall; the blocks are grey with open water on the left-hand ‘sea-side’ and saturated fill on the right-hand ‘fill-side’. When the wall is built too high, or a large hydraulic gradient is applied, the wall becomes unstable, either through sliding or toppling of a section. When this occurs, a red cell is displayed at the height at which the instability occurred (e.g. at the toe of the rotating section).

The model output gives physically rational results. Higher wall heights reduce stability; wider blocks give greater overturning resistance; lower friction angles for the fill increase lateral pressure; and the buoyancy-effect on the wall blocks reduces resistance to sliding and overturning. These results were encouraging, but the model needed to be verified.

Figure 3: Dry-build timber frame with partially constructed dry-build test.

A dedicated series of 'dry-build' tests using concrete blocks and granular fill (at nominal 1:30 scale) were used to test the stability model. These dry-builds were done in a timber 'box', as shown in Figure 3, which allowed testing the 2-dimensional assumption behind the analytical model. The characteristics of the blocks and fill used in the dry-builds were designed and measured to ensure that the analytical model inputs were ‘tuned’ correctly, so that an accurate comparison between the two could be made.
Key measurements were block density, fill density (rock density and fill porosity measurements), inter-block friction coefficient (pull-tests), and friction angle (estimated through angle of repose tests). The dry-builds were constructed one row at a time, slowly and carefully placing the fill as the walls got higher, until collapse. Initial dry-builds showed significantly higher stability than predicted by the analytical model; 20 rows high at failure in the dry-builds versus 11 in the model. This was due largely to jamming, or arching, across the wall. As the wall grew higher, it started to arch across the frame, and in doing so pushed the edge blocks outwards and against the side of the frame. The stiff timber frame provided lateral support, which pinched the blocks together and gave the wall greater strength against overturning. Arching of a wall in plan by up to 50mm (1.5m prototype) at the centre of the crest was observed. Once this was exceeded, the arch would snap through suddenly, the wall would unzip along the vertical centre line, and the structure would collapse entirely. A typical failed dry-build is shown in the left-frame of Figure 4, accompanied on the right by the same failed structure with the spilled core material dug out. This reveals the arched, but still intact, toe of the wall, about which the upper section toppled.

Figure 4: Failed dry-build. Left - immediately after collapse. Right - failed structure with spilled core removed to reveal arched (but still intact) wall toe.

It is noted that an overturning section in the model always rotated about the toe of the wall, or the bottom row. The difference of behaviour in the dry-builds may be in part due to the arching effect providing additional strength to the wall before a brittle collapse, but also perhaps due to the model's assumption of Mohr-Coulomb lateral earth pressure. This theory assumes that the force applied by the fill is horizontal, whereas strictly it acts slightly downwards and diagonally, activating friction between the fill and the wall. The downward, frictional component of this pressure tends to stabilise the wall by clamping the lower blocks more tightly into place, reducing the overturning, or disturbing, moment. This effect was investigated by Bray & Tatham (1992) and is illustrated in Figure 5. It is noted that for an internal friction angle of φ=25 and no wall friction angle (δ=0), the disturbing moment is equal to the restoring moment. When wall friction is increased to δ=φ, the restoring moment becomes 2.3 times the disturbing moment. It is uncertain whether the full effect of wall friction is activated on the scale of the dry-builds, but its omission in the analytical model along with the arching effect may explain partially why the model under-predicts wall stability and does not show toppling over a toe section.

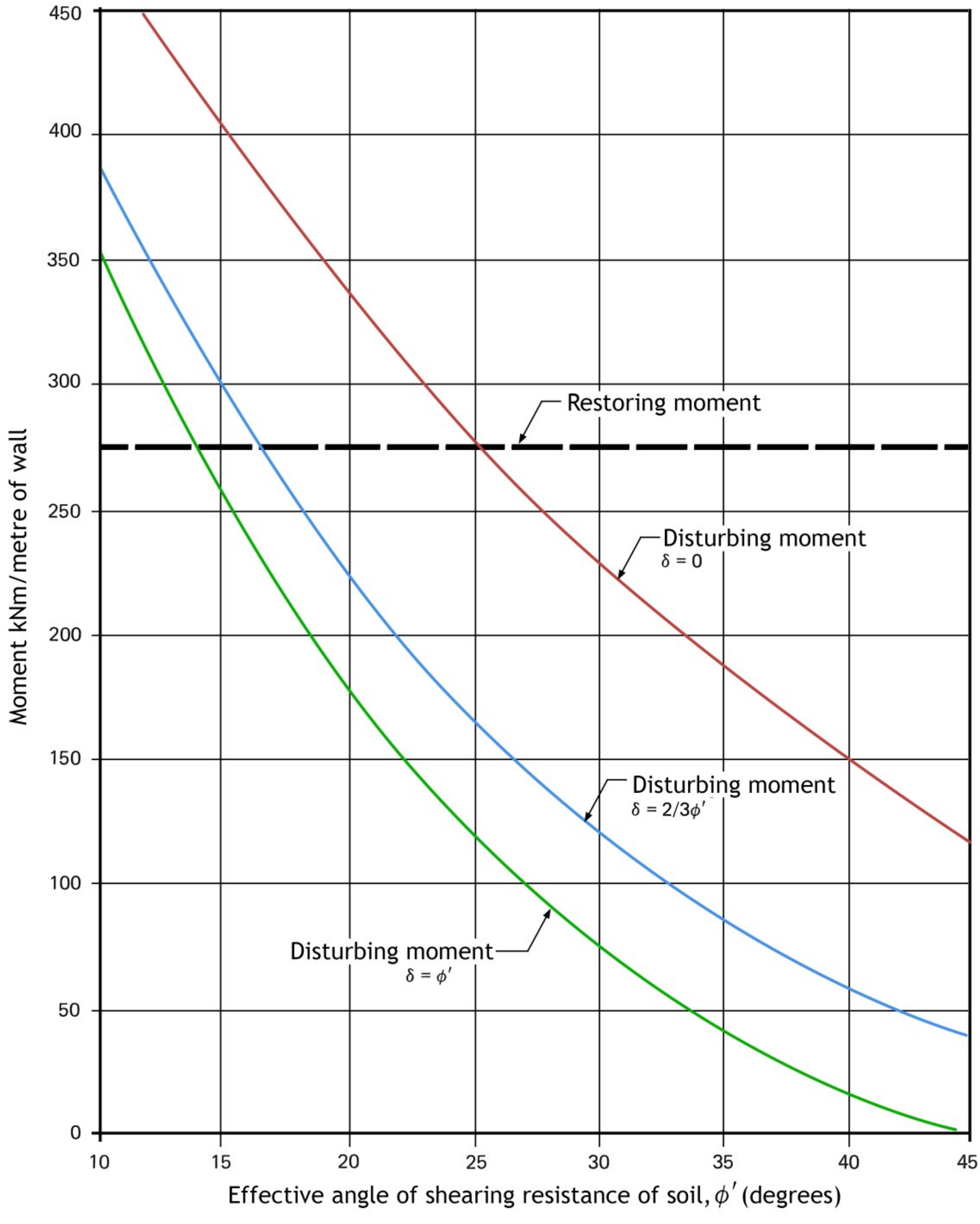

Figure 5: Effect of internal friction angle on overturning (disturbing) moment. Adapted from Bray & Tatham (1992).

In an attempt to reduce arching effect across the wall, PVC plates were installed on the inside faces of the timber frame to reduce friction between the blocks and the rough plywood sides. This reduced arching and brought the stability of the wall down to a closer agreement with the analytical model: 12-15 rows high at failure in the dry-builds versus 11 in the model. Tests were also performed with the blocks resting on their narrowest face, or ‘on-edge’. This reduced the stable wall height in the dry-builds by approximately 75-80%, from 12-15 blocks high to 3 blocks high. This sharp reduction matches results from the analytical model. The dry-build tests showed that the analytical model under-predicts wall stability by approximately 25-30%, even with PVC slip plates installed. This may be due largely to arching across the wall, which can be reduced in the model, but not eliminated. It was noted that this arching effect would likely be worse in the flume, where the walls are made of stiffer concrete which would provide more rigid lateral support. The dry-build tests did however confirm that a test structure could be built to at least 15 rows high. With a block height of 30mm and a geometric scale of 1:30, this corresponds to a prototype wall height of 13.5m, which matches well with the heights of typical blockwork breakwaters.

Results – collapse process

Tests in the flume were organised by structure and number. One complete set of tests on a particular structure is called a “Series”. Each Series comprised different test parts, progressing from Test Part 1 – the first waves – through to the final test where the collapsed breakwater reached a stable form. A summary is shown in Table 2 and a full description of series and test parts is given by Allsop *et al* (2017). An example test structure in the flume is shown in Figure 6.

Table 2: Summary of test series and parts.

Test Series	Rows in Wall	Wall / Crest Height [m]	Test Parts	Wave heights [m]
1	16	15.8, unarmoured	1-36	1.4–8.7
2	18	17.45, unarmoured	37-80	1.4–9.2
3	16	16.23, unarmoured	81-122	2.2–9.2
4	20	18.8, unarmoured	113-135	4.3–9.4
5	11+2	12.2, paved	136-169	1.3–5.8
6	11+2	12.2, paved	170-200	0.4–5.7

The test structure in Series 1 was constructed 16 blocks high, both front and back, bonded pattern. To limit mixing of the wall fill with the rubble foundation mound, a geotextile was laid over the mound. This proved to be of limited utility and was omitted in later tests. Starting with storm waves at H_s=1.4m, the front wall only began to fail when waves had been increased to ~ H_s=7.2m, when the crest course was knocked off, pushing blocks below backwards. Uplift pressures then lifted crest blocks out of the wall. This then led to increased local overtopping, in turn washing out fill material.

Multiple blocks were extracted seawards from the front face as overtopping and local wave pressures opened multiple gaps in the blockwork at a high level. Some courses resisted further due to arching of the blocks against the sides of the flume, clearly a model effect. As more blocks were extracted, more overtopping penetrated into the rubble fill, forcing out further blocks. Once the front wall had failed at about H_s=7.2m, wave backwash steadily washed fill out until the seaward blocks and rubble fill were graded to a relatively shallow slope angle down the front of the structure. With the front wall down, waves broke onto the new rubble beach, attacking the rear wall significantly less. The rear wall itself again showed considerable arching, probably leading to the wall staying intact to H_s=8.7m, much longer than might be expected without lateral restraint. Photos of this progressive type failure are shown in Figure 7. Measurements of front and rear wall crest heights are shown in Figure 8. It is noted that crest level is defined as the highest point of the structure; after the rear wall failed the core was often higher than the remaining wall sections.

Figure 6: Example test structure in flume.

Figure 7: Progressive failure of Series 1.

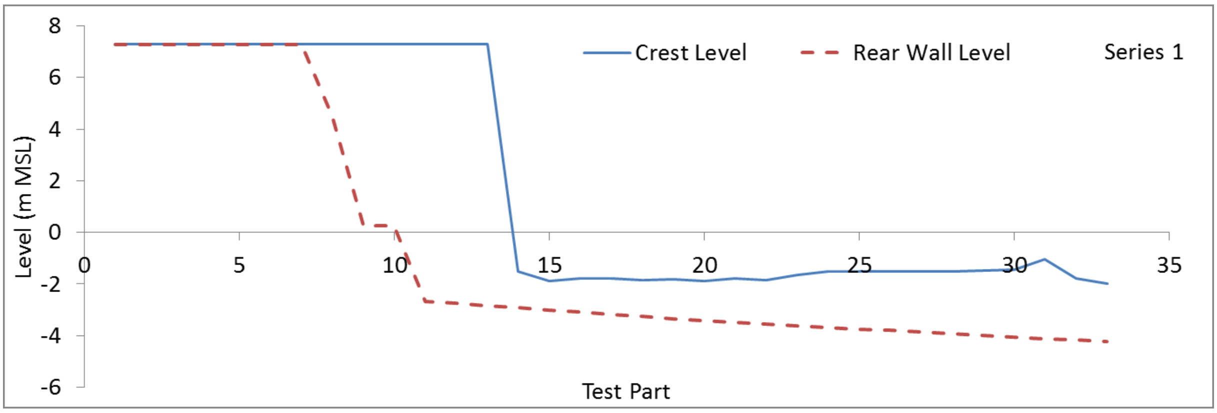

Figure 8: Progression of Test Series 1, test parts 1-33.

The wall in Series 2 was constructed 18 blocks high on the front, but fill and back wall were taken only to 16 courses. Blocks were again laid in bond. The geotextile in Series 1 was omitted as it was not useful. Sixteen test parts (each 500 waves) were required, failure occurring over test parts 48-52 (H_s=8.6-8.9m), illustrated in Figure 9.

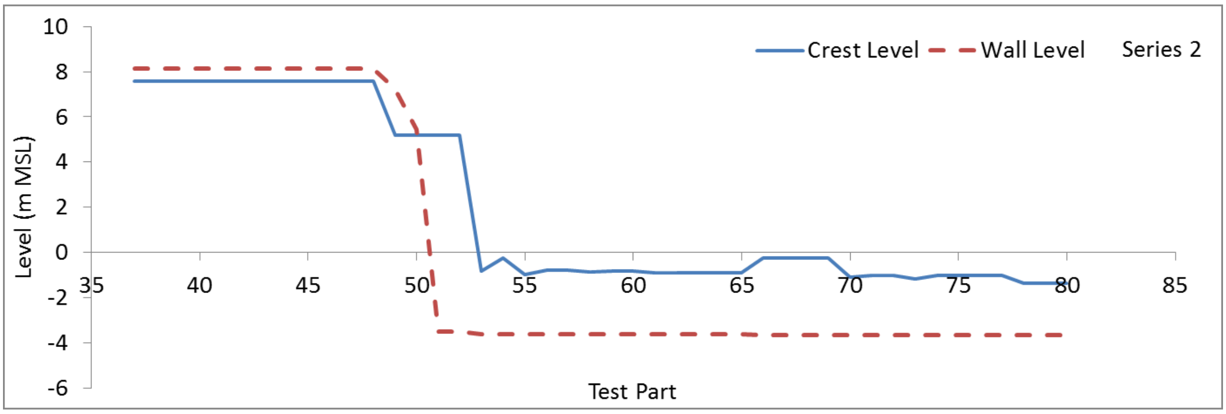

Figure 9: Progression of Test Series 2, test parts 37-80.

Wave conditions were increased to H_s=8.6m when some of the top course were flipped back by overtopping waves. As the test progressed, around 3 blocks were 'jerked' forward, allowing upward flows to lift and flip them backwards, as in Series 1. Once this key group of blocks had been removed, waves scoured out the remaining blocks and fill, exposing the rear wall to direct wave impact.

Again the speed of wall collapse was probably delayed by arching effects giving artificial restraint. As seen before, the front wall mostly collapsed seaward. The remaining courses of toe blocks acted to reduce erosion of fill material, a process that has been seen on site for collapsed walls. Measurements of rear wall and crest heights are shown in Figure 9. The eroded profiles (see Figure 10) were very similar to those seen for Series 1.

Figure 10: Post collapse view of Series 2, seaward side to right. Note the intact remnant of the wall toe.

The wall in Series 3 was built 15 blocks high in columnar bond with standard fill. A layer of fines allowed placing of a blockwork ‘capping’ layer bringing the section height to 16 blocks all the way across, see Figure 11. With the lower initial crest level, the wall in Series 3 suffered increased overtopping. At H_s=7.05m, the front edge of the capping layer began to lift by up-rushing waves. This caused the crest protection to deform, continuing until the row had been either thrown backwards onto the structure or washed forwards creating a gap in the front of the wall. Once this gap had been opened, waves were able to penetrate under the cap, washing individual blocks off relatively quickly. This allowed waves to pull the fill and front wall down within 500 waves, forming a profile similar to those seen in Series 1 and 2. Measurements of rear wall and crest heights are shown in Figure 12.

Figure 11: Series 3 with protected crest.

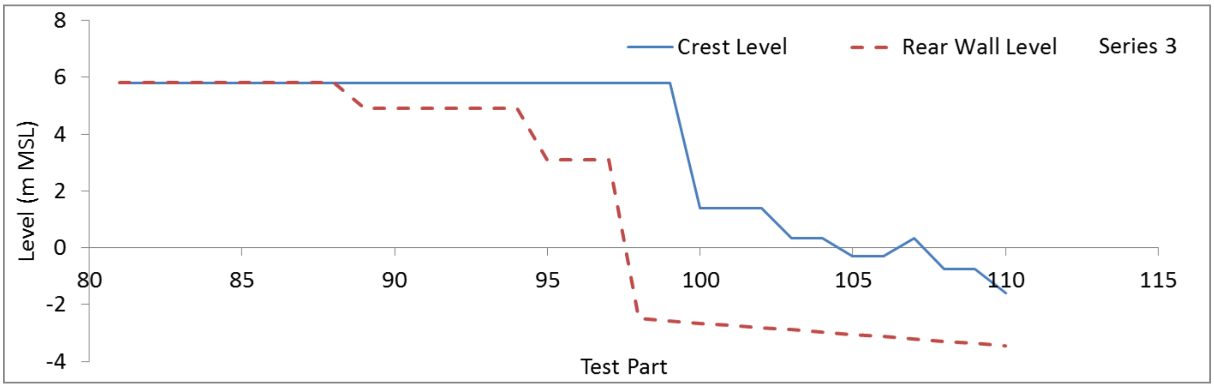

Figure 12: Progression of Test Series 3, test parts 82-110

Series 4 was built to a height of 20 blocks in columnar pattern, with PVC side panels to reduce end restraint. At H_s=6m the top layer began to show movement, with some blocks pushed backwards and one in the second row being extracted forwards. Soon more blocks were extracted from the central columns, one or two rows down, allowing fill to be extracted and creating a sink-hole behind the inside face of the front wall. Once enough blocks had been extracted, a section of wall began tilting backwards, and some blocks then fell down into gaps where those had been extracted. This drop in height allowed waves to overtop and wash out loose blocks and fill.

The following tests (H_s=8m) steadily brought the front wall down course by course, washing the fill out with it. The rear wall failed in a more brittle mode, perhaps due to reduced arching in this build. More core was washed over into the rear face when it collapsed, and the rear wall collapsed more as a single unit in Series 4 than in Series 3. Measurements of rear wall and crest heights are shown in Figure 13.

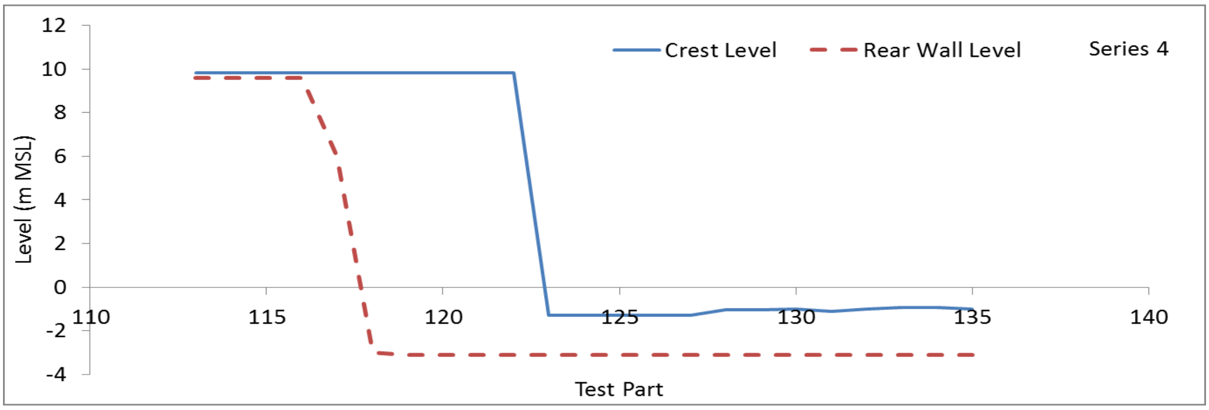

Figure 13: Progression of Test Series 4, test parts 113-135.

The walls in Series 5 were constructed only 11 blocks high, giving a crest level at +4.4m, and a wall width of 24.45m. Following experience in Series 3, the reduced crest level was expected to overtop more so the crest was protected by 3m x 3m aluminium slabs (ρ_c ~2.4t/m^3) equivalent to 0.1m thick. As common in breakwaters of this era, a parapet wall along the front edge was formed by two rows of wall blocks placed end-on; see Figure 14. Additional 'slip' measures were taken to reduce any artificial restraint to the blocks given by the sides of the wave flume.

Alongside the reduced crest level, a further change for Series 5 was to reduce the test water level by 1.5m to a nominal level of -1.5m. As before, wave conditions were ramped up at the 'storm' steepness (s≈0.06). In Test 138 at H_s=4.3m, some parapet wall blocks started to slide backwards. During Test 139 at H_s=4.9m more crest blocks were pushed backwards, thus allowing wave forces to lift the parapet wall blocks. During Test 141 (H_s=5.8m) half of the front wall collapsed around the hole left by two blocks extracted previously. During Test 142 (at essentially the same wave condition) the remaining half of the front wall collapsed. Most capping slabs were swept over the crest onto the rear foundation. The rear wall collapsed during Tests 143 – 145 (H_s ~ 5.5m). Measurements of rear wall and crest heights are shown in Figure 15.

Figure 14: Test section for Series 5, showing crest slabs and parapet wall.

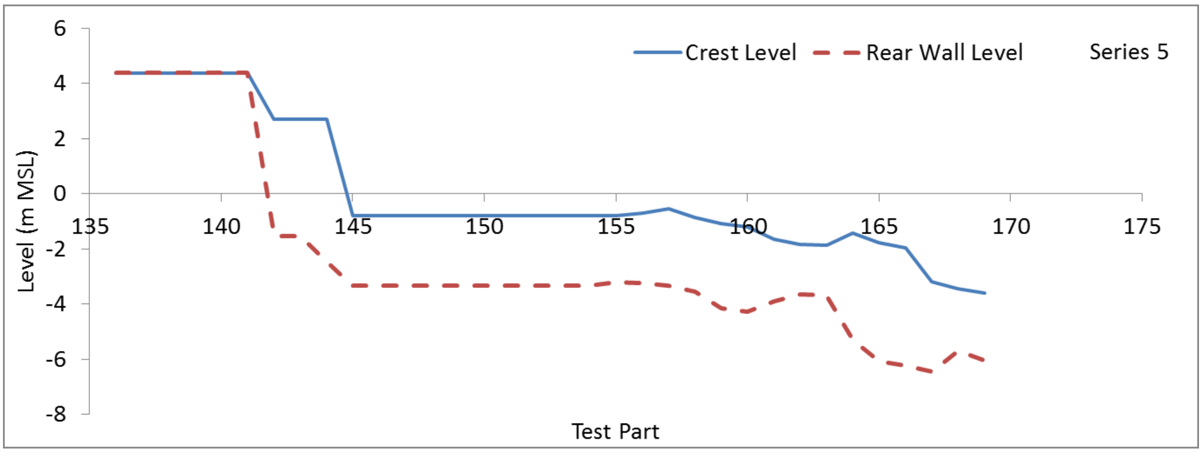

Figure 15: Progression of Test Series 5, test parts 136- 169.

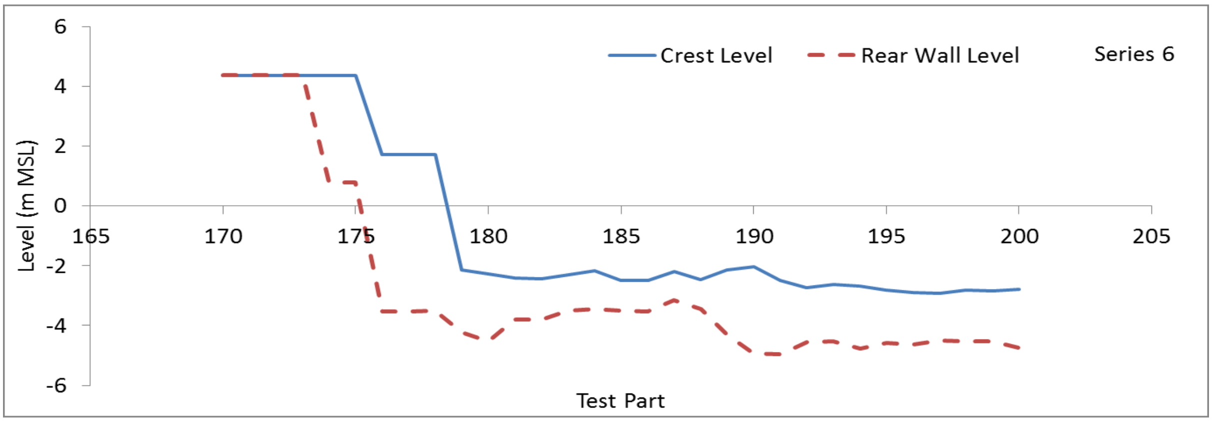

Figure 16: Progression of Test Series 6, test parts 170-200.

The test section in Series 6 was very similar to that in Series 5, again with main walls of 11 rows, a protected crest using large slabs, and a parapet wall of two layers of blocks placed end-on. Waves in Series 6 were increased from H_s=2m to H_s=5.7m when the first block was extracted from the front face, and crest protection slabs began to be washed off. Further tests at H_s=5.7m steadily extracted more blocks, particularly in rows just below the capping layer. During Test 173 a row of blocks was extracted and the parapet wall was pushed back so that it rested mainly on the crest slabs, rather than on the main front wall blocks. Further waves pulled blocks out seaward, causing a large hole in the centre of the section. This collapsed quite quickly, and the front wall was pulled over during Test 177 (H_s=5.5m). The rear wall failed faster than in previous tests, probably due to reduced arching. The top blocks were pushed back, then core material filled around them to form a fairly stable slope; see Figure 16.

Discussion of results

The review of historical documents, photos, and site visits gave ‘typical’ sections of old blockwork breakwaters. Section dimensions, block sizes, and core grading were scaled down and, with the use of the analytical model, enabled the initial design of a test structure. The model, although limited slightly by its simplifications, provided insight into wall failure mechanics once compared to dry-build tests in a timber frame. The comparison showed that 3-dimensional effects, specifically arching across the wall, lent significant extra strength to the wall. PVC slip plates were installed between the edge blocks and the frame walls, decreasing the friction between the two surfaces. This reduced arching and gave better agreement between the model and dry-build tests. These two experimental methods together aided the design of the test structure for the flume.

The flume tests showed higher wall stability than the dry builds, perhaps due to the rigid flume walls providing strong lateral support and enhancing the arching effect. PVC slip-plates alleviated this effect, but not entirely. Failure was less brittle and more progressive. Upper courses of blocks were jarred loose by wave impact, typically at H_s = 7-9m, and then overtopping and downfall pressures extracted them fully. This exposed core to direct wave attack, accelerating the front wall failure. The rear wall again showed more stability than expected, again due to arching. Once the walls had failed, the remains acted as a ‘non-engineered’ rubble mound with wall blocks scattered across the front and rear slopes, and visible remnants of intact wall toes. Despite the simplified geometry of the structures and the lack of degradation and settlement effects, the test structures in the flume showed encouragingly sensible failure mechanisms.

Acknowledgements

The studies described here are partly funded by ICE Research & Development Enabling Fund (ICE R&D) as project 1315, and in part from PhD research by the second author. Further support for the testing was given by Nick Hanousek (Industrial Trainee to HR Wallingford from Cardiff University), HR Wallingford (new wave paddle), Instrument Support (wave measurement and profiler equipment), Paul Tong (rock sorting), and Clive Rayfield and team. Dr Stephen Richardson was Project Director for HR Wallingford. Support from University of Edinburgh (Professors Tom Bruce and David Ingram) for the second author’s PhD studies are gratefully acknowledged.

References

Allsop, N. (2009). *Historical Experience of Vertical Breakwaters in the United Kingdom.* Proc ICE Conf. on Coasts, Marine Structures & Breakwaters. Thomas Telford. London.

Allsop, N., Pearson, A., & Bruce, T. (2017). *Orphan breakwaters - what protection might be given when they collapse?* Paper 135 to Coasts, Marine Structures and Breakwaters 2017. Liverpool.

Allsop, W. (2017). *Alderney and Jersey "harbours of refuge" - what was built, why did they fail, and what has remained?* Paper 107 to Coasts, Marine Structures, and Breakwaters 2017, Liverpool.

Bray, R. N., & Tatham, P. F. (1992). *Old waterfront walls: management, maintenance and rehabilitation*. CIRIA / E & FN Spon. London:

British Standards Institute. (2010). *BS 6349-2:2010: Maritime works - Part 2 - Code of practice for the design of quay walls, jetties and dolphins*. BSI Standards, London.

Williams, J. (1878). *Whitehaven harbour and dock works*, Proceedings of the Institution of Civil Engineers, Vol. 55. ICE. London.

Orphan breakwaters – what protection is given when they collapse?

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Abstract

Around the UK, many coastal harbours have reduced in importance and/or lost the original sources of income against which to defray maintenance or refurbishment. Their breakwaters may however still protect harbour-side properties against wave overtopping, and thus flooding. This paper presents results from an exploratory study to identify how blockwork breakwaters common in many smaller UK coastal harbours may collapse due to storm action, and in this paper, how much wave protection is given by collapsed breakwaters. The companion paper by Pearson & Allsop (2017) describes initial work to estimate the failure of blockwork walls, and presents results of wall collapse tests.

1. Introduction

Background

There is a long tradition in the UK of coastal towns or country estates constructing their own small harbours for trade and/or to shelter fishing boats. Such harbours were particularly needed on 'rocky' coastlines where the local topography / geology hindered construction of roads or railways. In the expansion of harbour construction around 1770-1880, many such harbours were protected against wave action by a breakwater (or multiple breakwaters) to reduce wave agitation, thus protecting quays, cargo handling facilities, and storage areas. These breakwaters were often formed by two walls of dressed stone (later concrete) blocks with random rubble or sand infill.

Figure 1: Wave attack on the Heugh breakwater at Hartlepool

For many such harbours, maritime incomes have since abated, especially with the movement of trade to rail or road, and with the diminution of fishing fleets. Many small harbours have therefore been left with little or no income against which to defray costs of maintaining / repairing their breakwater(s). If the only beneficiaries of wave protection from these 'orphan' breakwaters were the original commercial operations, this lack of resources might have relatively little consequence. But, in the decades since their original construction, the areas protected from wave action have been increasingly adopted for commercial and/or residential purposes. A potential problem then arises from the absence of funding for maintaining the breakwater, to be set against the risk of structural failure or damage. If the 'orphan' breakwater were to collapse, the protection afforded to areas sheltered from direct wave action will reduce. This was discussed for Hartlepool (Fig. 1) by Hampshire *et al*. (2013). Even after collapse, some wave reduction will still be afforded by the relict structure. In assessing the overall degree of protection, the level of wave transmission over the collapsed structure will be key.

This project

The studies described here were (part) funded by, ICE Research & Development Enabling Fund (ICE R&D) as project 1315. The question to be addressed was – "how much flood or erosion protection would we still receive if these orphan breakwaters were to fail?" It was proposed to use simplified 2D hydraulic model tests to explore wave transmission over damaged / failed walls for a set of selected example failure cases. Those test measurements would then populate an appropriate empirical framework to answer the over-arching question posed above. (It is noted that the funding was to enable initial or exploratory studies. The resources available for these tests were therefore in the order of ~ 25% of those that would be needed for a more definitive study.)

2. Outline of This Paper

This paper starts with a short review identifying example breakwaters in the UK of potential interest, and discusses their main modes of failure. It then describes the design of the hydraulic model tests to simulate key parts of the failure / collapse process, and to measure the main hydraulic responses of the collapsed structures (wave transmission and reflections).

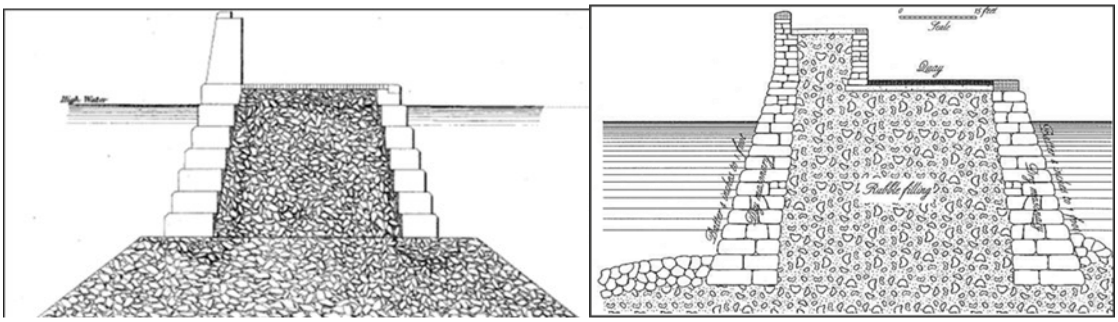

Figure 2: Blyth breakwater (left); Alderney breakwater (right)

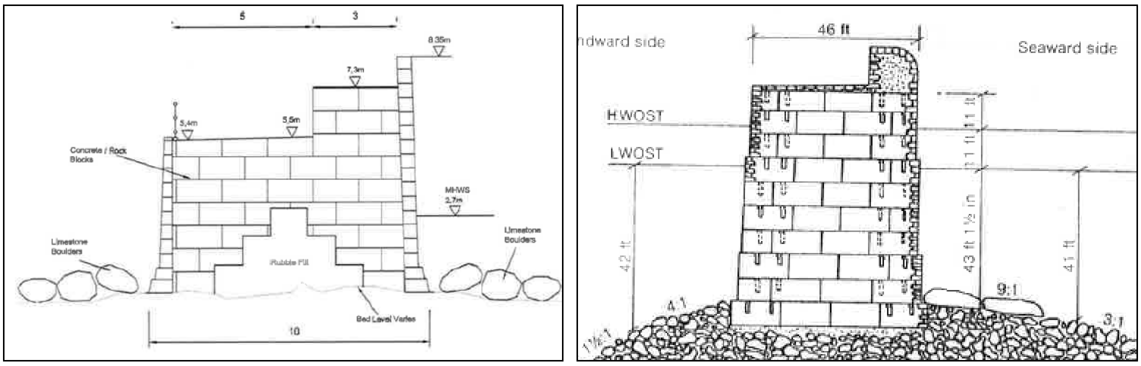

Figure 3: Hartlepool breakwater (left); Peterhead, after Buchan (1984) (right)

Six structures were tested in 200 separate Test Parts (see Table 1). The paper summarises both the collapse process, and measurements of wave transmission from which has been derived new guidance. The paper concludes with a discussion on how the results might be applied, and suggestions for further work to refine the results of this exploratory study.

3. Example breakwaters

Various blockwork breakwaters were reviewed to derive the simplified structures to be tested: Whitehaven, Blyth, Kilrush, Alderney / St. Catherine's, Peterhead, Hartlepool, Whitehaven. Historical records contain few construction details, but approximate values of key wall and block dimensions were extracted, and are summarised in the companion paper by Pearson & Allsop (2017).

The usually irregular blockwork, heterogeneous fill, and complex wave-structure interactions make stability assessment of these breakwaters difficult. Over the last few decades, physical modelling and case studies have progressed understanding of failure modes, structural resistance, loads and certain failure catalysts. Few design and analysis methods were available during the period of construction of these breakwaters. Lessons learned from previous failures typically guided the design process, but the causes of such failures may not have been correctly identified. Failure modes summarised by Allsop (2009) are:

a) Sliding or overturning of a breakwater section as a single entity;
b) Global geotechnical failure of the foundation, destabilising the wall;
c) Removal of blocks from the wall, resulting in discontinuity, hence structural instability;
d) Local geotechnical failure of the mound, destabilising the wall and/or releasing fill.

Of these c) removal of blocks; and d) local geotechnical failure are more frequent in the UK.

Stability of the wall depends on self-weight of the blocks, friction between blocks, lateral earth pressure from the core, and hydraulic pressures. Waves raise the internal phreatic surface in the core. The increase in outward pressure can cause the entire wall, or just a section, to collapse.

Figure 4: Greve du Lecq, Jersey: (top) after collapse in 1879; (bottom) recently, 2014

The simplified spreadsheet model by Pearson & Allsop (2017) was used to predict wall failure for the design of the physical model. The spreadsheet model was intended to assess the stability of an arbitrary section of the wall using static forces based on the following:

a) Configuration, dimensions and density of the blocks, hence self-weight, and friction;
b) Core material characteristics, size and natural angle, hence lateral earth pressures;
c) Free-water surface elevation and phreatic surface elevation, hence hydrostatic pressures and buoyancy effects.

The spreadsheet computes each force acting on each block. It then assesses rotational and translational equilibrium of each block and set of blocks, starting from the top down. The objectives of initial calculations were to:

a) Decide on block dimensions for the model tests;
b) Choose block orientation;
c) Define the maximum stable wall height;
d) Indicate the stability of the wall under quasi-static hydraulic gradients across the wall;
e) Explore the influence of fill friction angle.

[The model described by Pearson & Allsop (2017) was NOT originally intended for generic analysis, but it was judged that it might merit further development if supported by the experiments.]

4. Design of the model tests

The model study was intended to identify how much (or little!) wave protection might be given by the remains of such a breakwater after collapse. The main issues are therefore: to what crest level might it be reduced after initial failure; and how large might the transmitted waves be? The experiments to be presented were designed to identify initial collapse, then measure wave transmission.

Six series of 2-dimensional (2D) model tests were conducted, each Test Series testing to collapse six simplified breakwater sections using blockwork walls and rubble fill. The structural configurations used in each of these six Test Series are discussed in the companion paper by Pearson & Allsop (2017).

In the initial Test Parts, Storm waves (s~0.06) were used to fail the structure. Then once collapse had been initiated (or for most cases – completed), subsequent Test Parts used Persistent waves (s~0.035), and then Swell waves (s~0.01) to measure transmitted (and reflected) waves. As well as the wave transmission performance, collapsed structure profiles might later be compared with the collapsed profiles of known breakwaters, e.g. Greve du Lecq (Figure 4).

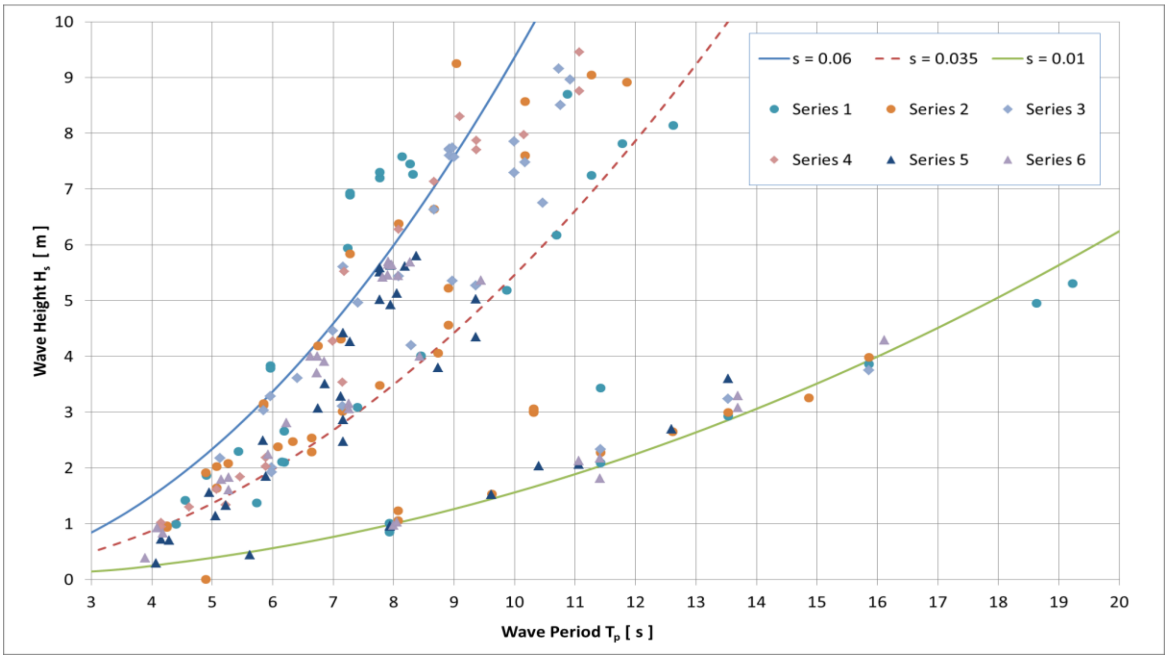

Figure 5: Test conditions, Storm (s=0.06), Persistent (s=0.035) and Swell (s=0.01)

Wave conditions at the chosen 1:30 scale are illustrated in Figure 5, extending up to H_s ~ 9.0m, typical of large North Sea storms. Three wave regimes were used, identified by their target wave steepness: Storm (s~0.06), Persistent (s~0.035) and Swell (s~0.01).

In each Test Series (see Table 1 below), wave conditions started at H_s ~ 1.0m, and were stepped up in each Test Part, most of which were run for 500 (or a few for 1000) waves. The shorter duration was generally used for the initial wall failure tests using waves of s~0.06. The shorter duration was based on testing of wave overtopping by Reis *et al* (2008), and later Romano *et al* (2015), both of which suggested that 500 waves may be sufficient, even for the highly non-linear response of peak wave overtopping volumes.

The walls in these models all used blocks equivalent to 4.2m x 2.1m x 0.9m. (Again we have used prototype values at Froude scale of 1:30 so that the dimensions and wave conditions are easier to compare with example structures.) The blocks were cast from cement mortar to a nominal density of ρ_{cm} = 2.33t/m^3, equivalent in seawater to ρ_c = 2.4t/m^3. In most of these tests, the blocks were laid in a single skin with the long side across the test flume, and in height steps (rows or courses) of 0.9m. A few of the later dry-build tests (see Pearson & Allsop, 2017) explored the stability of blocks on edge.

The preliminary dimensions of the test structure cross-section are ~13.5m high, 30m wide (and 30m across the flume). The 'typical' ratio of blocks to fill was 35% to 65%. The block sizes correspond well with those at Peterhead (Figure 3) and moderately with those at Blyth. The smaller blocks in the other breakwaters reviewed by Pearson & Allsop were less well represented by these large block sizes.

Table 1: Summary of structures, Test Series and Test Parts

Test Series	Rows	Wall height / Crest	Test Parts	Wave heights	Wave Type
1	16	15.8m unarmoured	1 - 14	1.4 – 8.7m	Storm, Persistent
			15 - 24	1.0 - 8.1m	Persistent
			25 - 36	0.8 – 5.3m	Swell
2	18	17.45m unarmoured	37 - 52	1.4 – 9.2m	Storm
			53 - 60	0.9 – 4.1m	Persistent
			61 - 80	1.1 – 5.2m	Swell (66 - 80 others)
3	16	16.23 paved crest	81 - 100	2.2 – 9.2m	Storm
			101 - 110	1.9 – 5.3m	All types
4	20	18.8 unarmoured	113 - 122	4.3 – 9.4m	Storm
			123 - 135	0.9 – 4.9m	Persistent
5	11 + 2	12.2 paved	136 – 144	1.3 – 5.8m	Storm
			145 - 169	0.3 – 5.1m	All types
6	11 + 2	12.2 paved	170 - 181	0.4 – 5.7m	Storm
			182 - 185	0.9 – 4.0m	Persistent
			186 - 200	1.0 – 5.4m	All types

The model core material is widely graded with D_{max} ~ 1.5m, to ensure that the maximum rock size was smaller than a typical wall block. Recalling that a typical core will be formed by discarded wall stones, chippings, and natural tout-venant ('all-in') rubble core, the natural upper-limit is indeed the wall-block size, smaller than Peterhead and Blyth wall blocks.

In the wave flume, all blockwork walls were constructed on a mound from flume floor at -24m to a platform level nominally at -10m, example shown in Figure 6. All levels are expressed relative to nominal still water level of 0.0m.

Figure 6: Example test section construction (Series 4)

The test programme was formed by six Test Series, each run on slightly different structures. The early test structures suffered from excessive levels of restraint from the side walls, delaying the onset and process of wall collapse. It is not however expected that these effects will significantly impact on the wave transmission results.

5. Results – collapse process

[Details of the collapse process are given in Pearson & Allsop (2017).]

The test structure in Series 1 was 16 blocks high, bonded pattern. Under waves, the seaward wall began to fail at around H_s=7.2m. Uplift pressures lifted crest blocks off the wall. Increased local overtopping then washed out fill. Multiple blocks were extracted seawards from the front face as local wave pressures opened gaps at a high level, but some courses arched against the flume sides. Once the seaward wall had failed, fill washed out until the seaward blocks and rubble fill were graded to a relatively shallow slope angle down the front of the structure. Waves broke onto this rubble, attacking the rear wall less. The rear wall itself again showed arching leading to the wall staying intact much longer than might be expected without lateral restraint.

Again in Test Series 2, wave conditions were increased to H_s~9m when some top course blocks flipped back under overtopping. Three blocks moved outward allowing upward flows to lift them. Once these blocks had been removed, waves scoured out the remaining blocks and fill, exposing the leeward-wall to direct wave impact. Again the speed of wall collapse was probably delayed by arching giving artificial restraint.

As before, the seaward wall mostly collapsed seaward. The remaining courses of blocks acted to reduce erosion of fill material, a process that has been seen on site for collapsed walls. The eroded profiles (Figure 7) were very similar to those seen for Series 1.

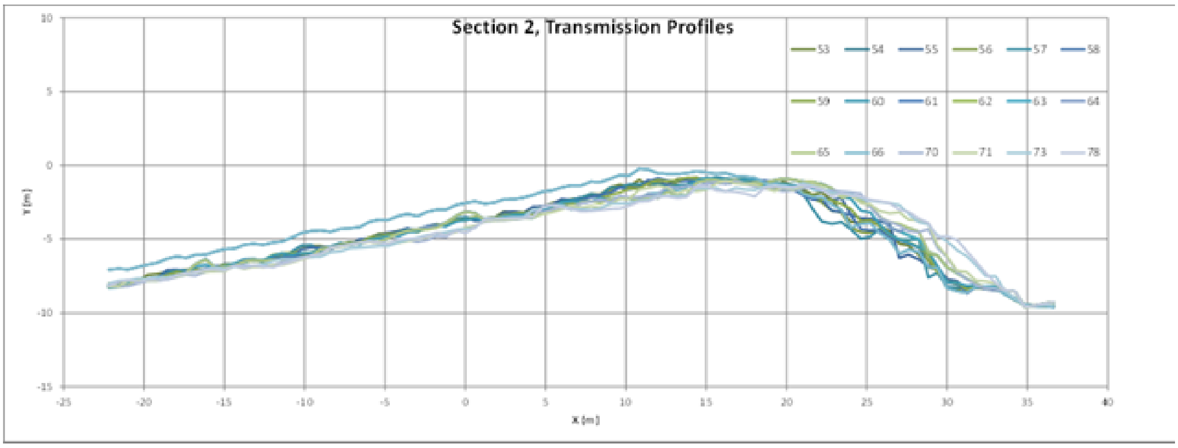

Figure 7: Test Series 2, post collapse profile

The wall in Series 3 was built 15 blocks high in columnar bond. A blockwork ‘capping’ layer brought the section to 16 blocks. With the lower initial crest level, the wall in Series 3 suffered increased overtopping. At H_s=7.05m, the seaward edge of the capping layer began to lift causing the crest protection to deform. Once gaps had been opened, waves penetrated the cap, washing blocks off quickly. The collapse of the front wall formed a profile similar to seen in Series 1 and 2.

Series 4 was 20 blocks high in columnar pattern. At H_s=6m the top layer began to show movement, with some blocks pushed backward and one extracted forwards. Soon more blocks were extracted from the centre, allowing fill to be extracted, with a sink-hole inside the front wall. Soon a section of wall tilted back and blocks fell into gaps where others had been extracted. This allowed waves to overtop, washing out loose blocks and fill. The following tests (H_s=8m) brought the front wall down course by course. The leeward wall failed in a more brittle mode, perhaps due to reduced arching in this build. More core was washed over backwards when the rear face collapsed.

The walls in Series 5 were constructed only 11 blocks high, giving a crest level at +4.4m, and a wall width of 24.45m. Following experience in Series 3, the crest was protected by 3m x 3m slabs (ρ_c ~2.4t/m^3) equivalent to 0.1m thick. As common in breakwaters of this era, a parapet wall along the seaward edge was formed by wall blocks placed end-on. Additional 'slip' measures were taken to reduce artificial restraint to the blocks from the sides of the wave flume.

Alongside the reduced crest level, Series 5 was tested at a water level -1.5m. As previously, wave conditions were ramped up along the ‘storm' curve ($s \approx 0.06$). Again the failure stated by loss of parapet wall blocks. At H_s=4.9m more crest blocks were pushed backwards, allowing waves to lift the parapet wall blocks. For H_s=5.8m, half of the seaward wall collapsed around a hole, then the rest of the front wall collapsed. Most capping slabs were swept over the crest onto the leeward foundation.

The test section in Series 6 was very similar to that in Series 5, again with 11 rows, a protected crest using large slabs, and a parapet wall of 2 blocks end-on. Waves in Series 6 were increased from H_s=2m to H_s=5.7m when the first block was extracted from the front face, and crest slabs began to be washed off. Further tests at H_s=5.7m steadily extracted more blocks, particularly in rows just below the capping layer. Then a row of blocks were extracted and the parapet wall was pushed back. Further waves pulled blocks out seaward, causing a large hole in the centre of the section. This collapsed quite quickly, and the front wall was pulled over. The leeward wall failed faster than in previous tests, probably due to the reduced arching. The top blocks were pushed backward, then core material filled around them to form a fairly stable 'armoured' slope.

6. Results – wave transmission and reflections

Each Test Series used wave conditions following the three mean wave steepnesses in Figure 5. Wave gauges seaward of the test section quantified incident and reflected waves, and another behind measured transmitted waves. Reflected and transmitted wave heights were then divided by the incident height to give reflection and transmission coefficients C_r and C_t respectively. Initially, C_r and C_t were plotted series by series to give indicative 'histories' of each test. These graphs may be read with the 'damage progression' graphs given by the 'sister' paper by Pearson & Allsop.

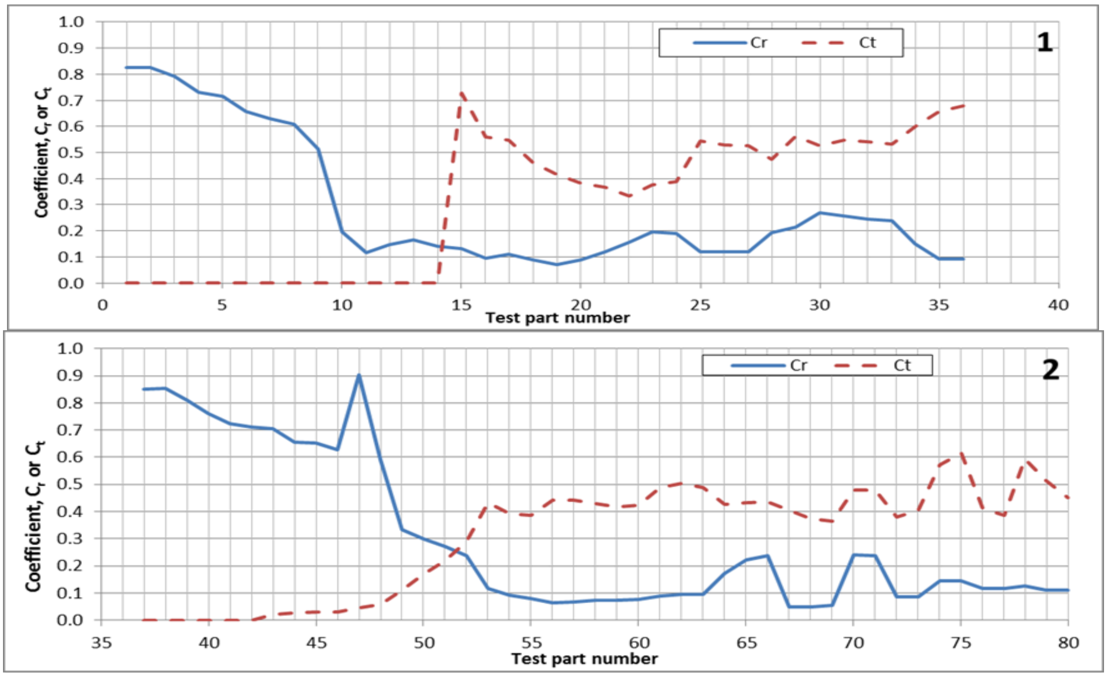

Figure 8: Reflections and transmission, Series 1 (top) and Series 2 (bottom).

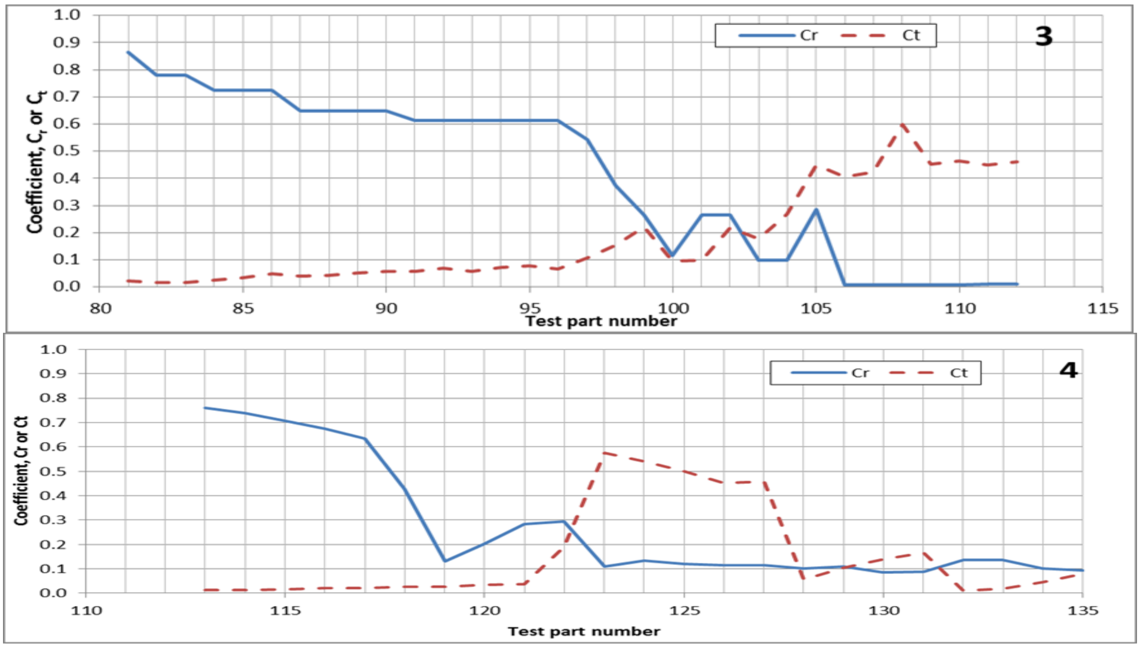

Figure 9: Reflections and transmission, Series 3 (top) and Series 4 (bottom).

For Series 1, the reductions in C_r in Figure 8 suggest that damage to the front wall started relatively early. Conversely C_t depends strongly on failure of the rear wall. The sudden increase of transmission in Test Part 14 correlates well with observations of the rear wall failure. Thereafter, reflections continue at a relatively low level, $C_r < 0.25$, whilst transmission stabilised at $C_t \sim 0.55$.

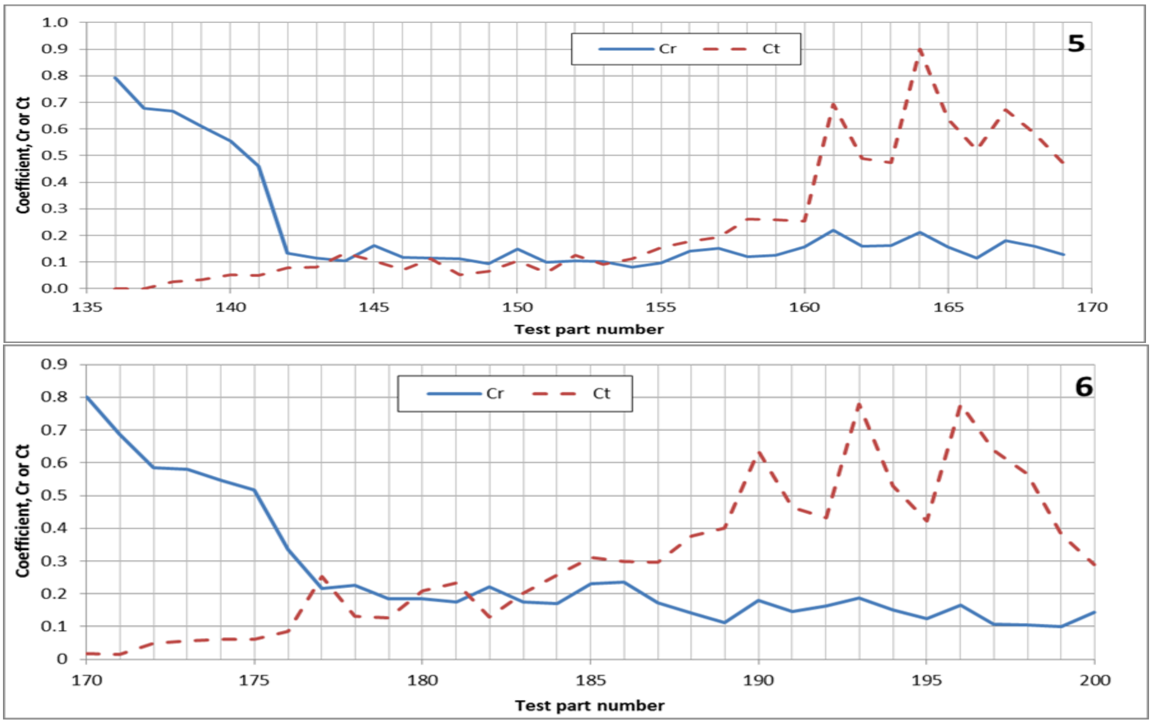

Figure 10: Reflections and transmission, Series 5 (top) and Series 6 (bottom).

In Series 2 (Figure 8) the process of wall failure was more abrupt with increased transmissions around Test Part 52 following soon after the fall in reflections as the rubble slope develops from the debris of the front wall collapse in Test Part 49.

The wall failure in Series 3 was more gradual with reflections reducing as waves overtopped more (Figure 9). The rear wall failed incrementally, with transmission increasing as reflections reduced. This process was repeated in Series 4 (also in Figure 9), again with a lag in lower reflections as the front wall fails before the rear wall.

As expected, Series 5 and 6 behaved similarly. Reflections (Figure 10) fell rapidly with increased overtopping, then as the vertical front face degraded to a slope. Again the increase of transmission (also Figure 10) was delayed until the rear wall collapsed, then varying with the test wave condition.

7. Discussion on use of these results

The form of presentation of wave transmission measurements in section 6 links C_r and C_t to the damage process, but this form is not easily used to predict transmission. Values of transmission coefficient C_t have been plotted against the simplest dimensionless freeboard, R_c/H_{si}, in Figure 11, using mound crest levels from the profile measurements and the test water level to calculate R_c. Results for each of the test steepnesses have been plotted separately in Figure 11. (An attempt was made to explore whether an alternative approach by Powell & Allsop reduces scatter by including wave steepness where C_t is plotted against $R^* = (R_c/H_{si})/(\sqrt{s}(2\pi)$. The improvement was slight, judged not worth the increase of complexity in the plotting parameter.)

As presented in Figure 11, the results show that there were very few instances when the crest of the damaged mound fell below $R_c/H_s \sim -2$, or where wave transmission exceeded $C_t \sim 0.6$. The few instances of higher C_t arose from inherently smaller wave conditions. At first pass therefore, these results suggest that these two limits might safely be used in any initial assessment.

8. Wave transmission prediction

The results in Figure 11 do not however allow easy prediction of C_t. The study results have therefore been combined in a single overall graph in Figure 12, to which has been added the prediction line given in the Rock Manual, plotted as Series 17.

That prediction line appears however excessively pessimistic for these test results, so a revised set of prediction lines have been fitted to the data from this exploratory study, plotted in Figure 12 as Series 20:

$-4 < R_c/H_s < -1.6$ $\quad C_t=0.8$

$-1.6 < R_c/H_s < -0.7$
$\quad C_t=0.32 – 0.3\ R_c/H_s$

$0.7 < R_c/H_s < 3.0$ $\quad C_t=0.1$

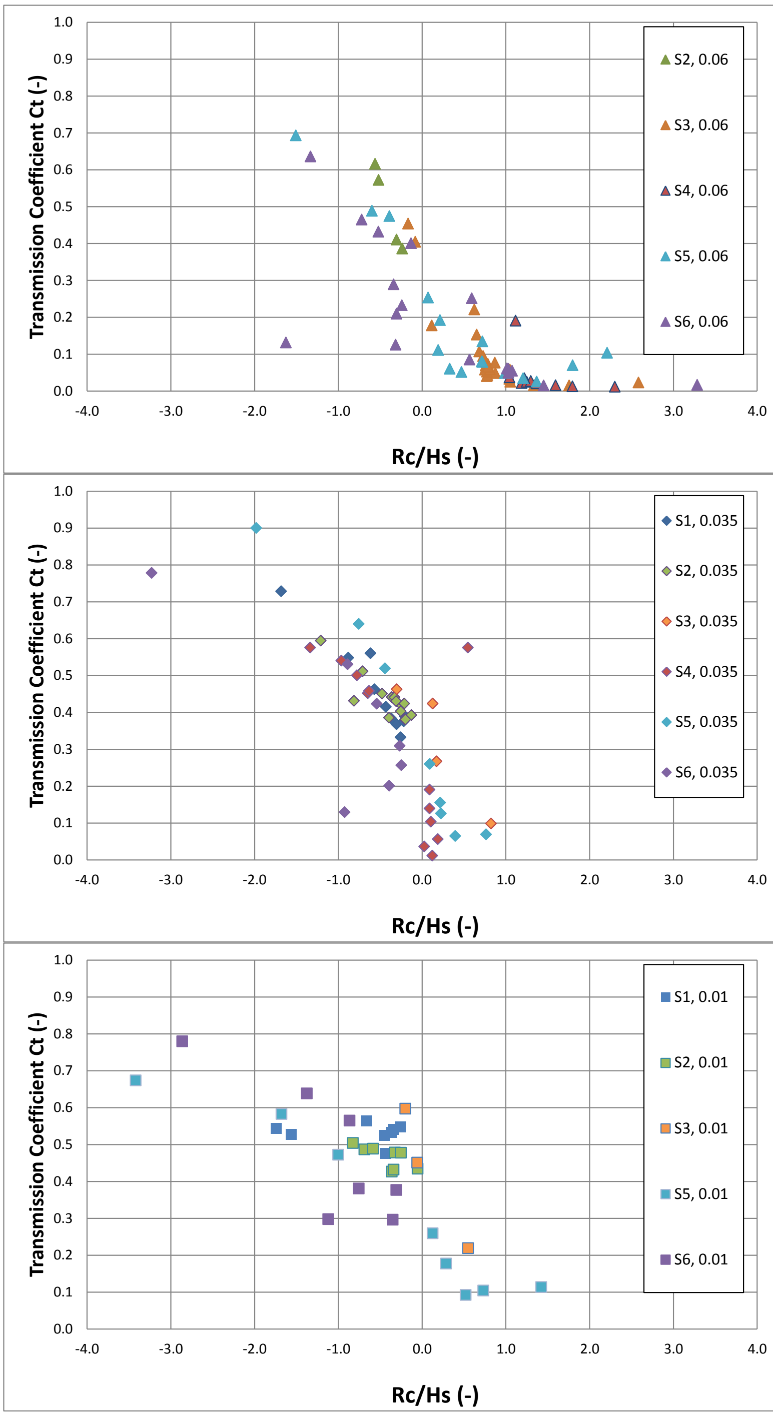

Figure 11: Wave transmission, C_t vs R_c/H_{si}: s= 0.06; s=0.035; s=0.01

9. Further work

These simple exploratory tests have been more successful than anticipated in reproducing many of the failure processes, even if some tests were influenced by excessive wall restraint delaying failure. Whilst most of the model 'arching' effects were reduced in Series 5 and 6, it is however probable that the results may have been (in part) influenced by the (large) size of blocks used. Further tests with smaller blocks might explore this effect. It would also improve confidence if future tests (with smaller wall blocks) were to explore the influence of tidal variations on profile recession, and hence related transmission. Even so, it seems likely that the lower level of transmission discussed in section 8 above relative to the Rock Manual prediction is realistic.

Acknowledgements

The studies described here were supported by ICE Research & Development Enabling Fund (ICE R&D) as project 1315, and in part from PhD research by the first author. Additional support was given by Nick Hanousek (Industrial Trainee to HR Wallingford from Cardiff University), HR Wallingford (new wave paddle), Instrument Support (wave measurement and profiler equipment), Paul Tong (rock sorting), Clive Rayfield and team. Elysia Ward (student) and Dr Dave Simmonds (Plymouth University) are thanked for access to Elysia's exploratory project thesis. Support from University of Edinburgh

(Professors Tom Bruce and David Ingram) for the first author's PhD studies are gratefully acknowledged. Dr Stephen Richardson was Project Director for HR Wallingford.

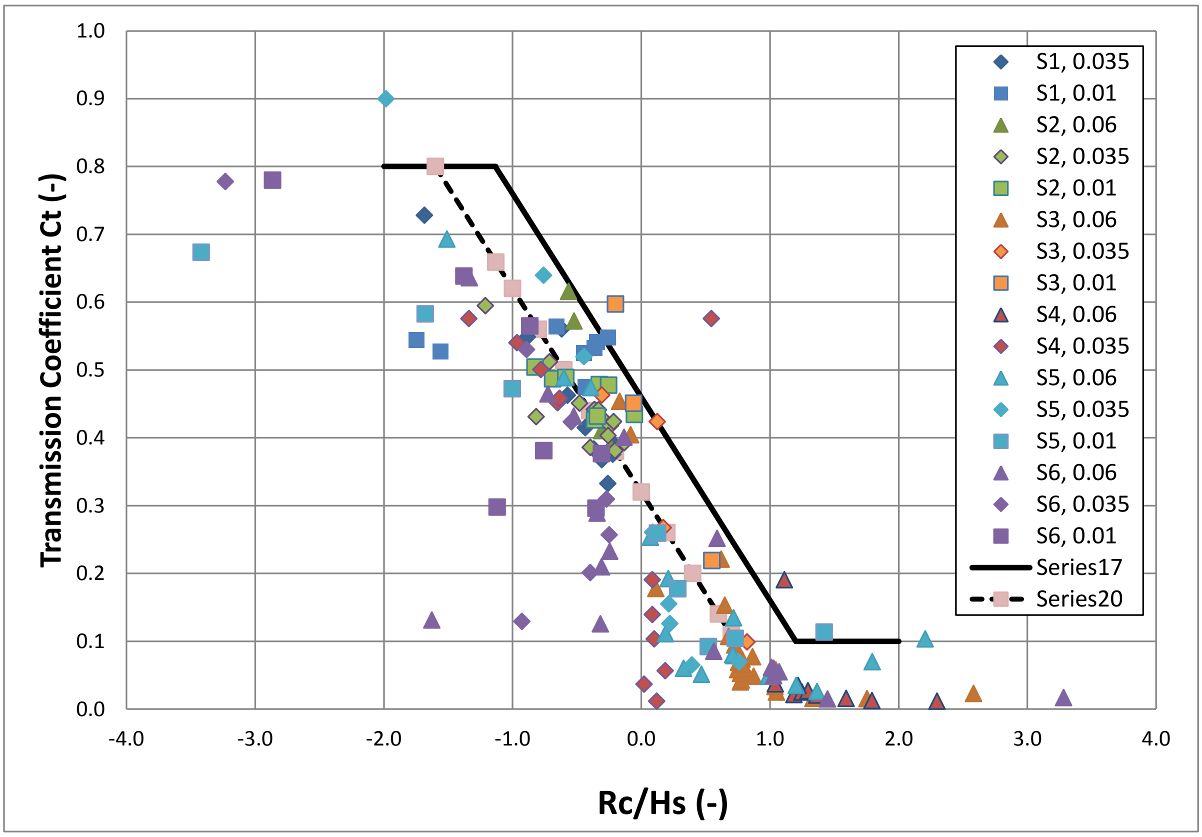

Figure 12: Prediction graph for C_t showing new prediction lines

References

Allsop N.W.H. & Bray R.N. (1994) *Vertical breakwaters in the United Kingdom: historical and recent experience*, Proc. Int. Workshop on Wave Barriers in Deep Waters, pp76-100, PHRI, Yokosuka.

Allsop N.W.H. (2009) *Historical Experience of Vertical Breakwaters in the United Kingdom*, Proc ICE Breakwaters Conf., Edinburgh, Sept 2009, ISBN 978-0-7277-4129-5, publn. Thomas Telford, London.

Allsop W & Pearson A (2016) *Failures of historic vertical blockwork breakwaters - 2D physical model inception report*, Report CAS1222-RT001-R02, HR Wallingford.

CIRIA; CUR; CETMEF (2007) *The Rock Manual. The use of rock in hydraulic engineering* (2nd edition) Publication C683, CIRIA, London

Grauw, A de (2013) *Stability of overtopped and submerged rubble mound breakwaters*, Note on model testing on Ancient Breakwaters web-site.

Hampshire S, Turner R, Hancock D & Guthrie G (2013) *In a Heugh – the strategic influence of a breakwater*, Proc ICE Breakwaters Conference, ICE Publishing, ISBN 978-0-7277-5975-7, pp188-198.

Pearson A & Allsop W (2017) *Exploring structural stability of old blockwork breakwaters*, Paper 140 to ICE Breakwaters conference, ICE Publishing.

Powell K A & Allsop N W H. (1985) *Low crest breakwaters, hydraulic performance and stability*, Report SR 57, Hydraulics Research, Wallingford.

Reis MT, Neves MG, & Hedges TS (2008) *Investigating the lengths of scale model tests to determine mean wave overtopping discharges*, Coastal Engineering Journal, World Scientific, 50(4), pp441-462.

Romano A, Bellotti G, Briganti R, & Franco L (2015) *Uncertainties in the physical modelling of the wave overtopping over a rubble mound breakwater: The role of the seeding number and of the test duration*, Coastal Engineering, 103, pp15-21.