Coasts, Marine Structures and Breakwaters 2023

Resilience and adaptability in a changing climate

Edited by Kevin Burgess

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Editor’s Preface

These proceedings present nearly 90 peer reviewed papers and discussion from the ***12th ICE Coasts, Marine Structures & Breakwaters*** conference held in Portsmouth in April 2023. Papers were presented in 27 sessions, mostly running in parallel but also with a plenary on each of the three days of the conference.

The ‘Breakwaters’ conference series started formally in 1983 after a plethora of breakwater failures of the late 1970s, with the objective to learn from such failures and evolve the science. Some 40 years on, we were reminded of those failures, with a challenge put out in the opening session on the robustness and resilience of single layer concrete armour units, followed later by talks on the analysis, design, construction, tools and techniques being continually developed to further improve our knowledge and application of those.

The conference has grown and broadened immensely since 1983, as can be in the range of topics covered by the papers within these proceedings, and the authors and audiences are far more diverse. Engineers still, but bringing wider engineering skills and joined by colleagues engaged in environmental, ecological, coastal and public project management. We have done many things, but two we’ve continued to excel in at this conference is learning from history and continually worked on improving our techniques, and our understanding.

We also heard about the closeness between the principles of this conference series, and those that underpinned the founding of the ICE nearly 2 centuries ago. In particular, it served to remind us that it is not just the papers that matter, but it’s also the quality of the discussion and debate that follows. Perhaps uniquely these days, we still capture the conference discussion and include the questions and answers in these published proceedings as this can often be as illuminating and informative as the papers themselves.

The work and research presented at Breakwaters has evolved immensely over time, as has the name, size and format of this event to accommodate those advances and breadth of topics to be discussed. But we also continue to make sure we do not lose sight of its roots and what made, and continues to make, it a special and leading conference for those involved in our industry. As previous editions have proven, I am certain that these latest proceedings will again become an essential reference for many years to come.

Kevin Burgess
Editor & Chair,
ICE Breakwaters Conference Organising Committee

Plenary 1

1.1 Lessons from old breakwaters
1.2 Protecting Portsmouth: The UK's Island City

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LESSONS FROM OLD BREAKWATERS

N.W.H. Allsop[1] and T. Bruce[2]

[1]Director, William Allsop Consulting Ltd

[2]Professor, Institute of Energy Systems, University of Edinburgh

ABSTRACT This paper summarises lessons learnt from research on British breakwaters built between 1670 and 1910. Drawing on the comprehensive archive of papers, and discussions in ICE Proceedings, and supported by analysis of the failures of breakwaters at Alderney, Wick, and elsewhere, the research has distilled the key reasons for failures of many breakwaters in this period; and the reasons behind the success of those that survived, as highlighted by the outer harbour breakwaters at Dover.

1. Background

Over 300+ years, the UK has built many breakwaters. Some remain fully operational, some extant but damaged, others in varying levels of distress, some destroyed. This paper discusses example breakwaters from which it will outline key changes over 1670-1910. The paper will particularly note how changes to understanding, materials, and construction technologies, reduced incidence of failure. Selected stages in this research have been published in previous conferences, and/or in ICE proceedings[2, 3, 4, 6, 7].

The research has concentrated primarily on vertical, battered, or horizontally composite breakwaters, so no significant attention is paid to rubble mound breakwaters[1]. The thesis, and this paper, have been constrained to the period 1670-1910. The primary motivation for research was to identify key technical changes over that period that led to the success or failure of representative breakwaters, and therefore to reveal how designs changed during this period. Two sub-questions were:

a) why did Alderney breakwater keep failing, yet the breakwaters at Dover worked right from the start?

b) when these breakwaters do fail, how much wave protection might the collapsed wall and mound provide?

The start and end dates for the research were book-ended by construction of the breakwater at Tangier in the 1670s, ultimately abortive; and the structural, political, economic, and commercial success of the outer harbour breakwaters at Dover completed in 1910.

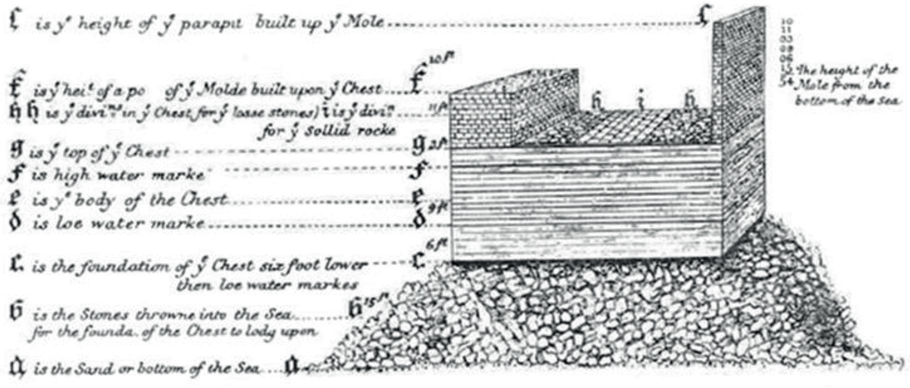
In analysing lessons learnt by engineers through the period of interest, it is helpful to take events sequentially where possible. It is however noted that developments in understanding of hydrodynamics, availability of new materials and equipment, and of engineering techniques, did not follow linear sequences, perhaps more a series of iterative spirals. But first the key problems are clarified; advances in understanding of hydrodynamics are described; as are improvements in construction; and advances in knowledge management.

An addition to the planned research was a series of physical model tests of idealised blockwork breakwaters[2]. These tests gave insights into failure processes and progression, correlating with examples from the historical review and site observations, and provided specific guidance on wave transmission over collapsed breakwater mounds[2].

2. Breakwater failures

The first breakwater reviewed here was at Tangier[3, 18], occupied by British forces 1663-1685. Initially constructed as a classic blockwork wall on a rubble mound, this breakwater was frequently damaged during construction, and took much longer and cost considerably more than budgeted. A solution to construction difficulties was afforded by innovative use of timber caissons copied from engineers at Genoa[19]. Construction with these caissons appears to have been entirely successful, but the harbour was abandoned in 1683 for political / military reasons, and departing British forces destroyed the breakwater to avoid it falling into the hands of potential enemies. Perhaps because it was far away, perhaps because the mechanisms for sharing engineering knowledge were not well developed, the lessons of the Tangier caissons were not learnt, and the experiment was not repeated in the UK possibly until deployment of the Mulberry Harbour caissons in the 1940s.

Figure 1 The Great Chest by Mr Shere, June 1677



After Routh, 1912[18]

After 1830, engineers involved in design and construction of harbours were faced by a UK government wanting to expand ‘harbours of refuge’[4]. Through the 1850s and 1860s, construction of the new Admiralty breakwater at Alderney suffered repeated damage, yet each design revision showed only partial success.[3, 5]. Papers to ICE were discussed by engineers perhaps attracted by potential contracts for further harbours of refuge to counter the threat of the French navy harbour Cherbourg[4], focussed on analysing the problems at Alderney, and promoting other solutions.

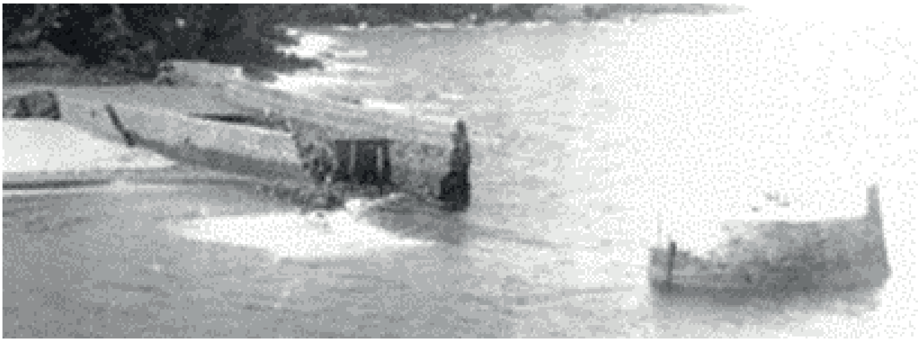
Figure 2 Use of timber cones at Cherbourg

Courtesy Alderney Museum

On Jersey in the Channel Islands, the breakwater at St Catherine's was completed in 1855 without incident, but the accompanying lee breakwater at Archirondel was abandoned at an early stage in 1849, and never proceeded further.

In Scotland, Portpatrick Harbour lost its mail contract in 1862, followed quickly (but probably unrelatedly) by collapse of the outer ends of both breakwaters. At Wick, Stevenson's new breakwater lost its outer end to storm damage in 1870 before construction was entirely complete[33], followed by collapse of most of the breakwater trunk. The failure here (explained by Allsop & Bruce, 2020[7]) was precipitated by wave breaking over the Crane Rocks over which incident waves will have broken violently onto the wall. The case study[1,7] calculated Factors of Safety against sliding of 0.6-0.8 under wave conditions of H_s=8-10m, not-unreasonable for a North Sea storm in outer Wick Bay.

Figure 3 Failure at Greve du Lecq, Jersey, 1884



Courtesy Jersey Harbour Master (1884)

Courtesy Allsop (2014)

On a much smaller scale, the breakwater at Skateraw (near Torness) collapsed sometime between 1855 and 1890, and that at Greve de Lecq (Jersey) in 1879 and again in 1885[1, 6]. At Greve de Lecq, an inspecting engineer[1] reported on failure of the pierhead in 1879 when he wrote: "*…the quality of the masonry is altogether wretched and worthless, and that if once holes were formed in the face … the collapse of the wall would only be a question of time, longer or shorter according to the severity of the gales.*" He later (1885) added *"…for at least 12 months past, considerable holes have existed in the lower party of this wall… It is quite certain that the immediate cause of the failure of the wall was that the waves by working through the holes in the face washed out the heart of the work and caused its collapse.*" So, a workmanship failure!

At Dover, construction of an extension to Admiralty Pier during the 1870s cost much more than expected, and the parapet wall was swept away in 1877[6, 34]. The reasons for failures were multiple, but the primary causes may be summarised:

- Weaknesses in the prediction of offshore or nearshore wave heights.
- Lack of understanding of effects on incident waves of natural shoals or foundation mounds, inducing wave breaking against the super-structures.
- Absence of wave load prediction methods.
- Difficulties in lowering the foundation level of the superstructure walls, leaving them in the 'danger zone' caused by impulsive breaking.
- Weaknesses of (stone) blockwork walls under wave loads, compounded by the complete absence of any method of analysis.

3. Improvements in prediction methods

3.1 Predicting waves

In his 1874 handbook, Stevenson[8] discusses the simple rule developed in 1852 to predict wave heights (H in feet) by the square root of the fetch (in nautical miles) multiplied by a coefficient depending on wind strength. Stevenson also notes that Hawksley had published a formula in 1861 of similar form, but gave "*much greater*" wave heights. In his manual, Vernon Harcourt[9] offers no significant improvement. Shield[10] (1895) discusses more wave height examples but includes a modified version for shorter fetches. Throughout this period, designers would never have had reliable wave predictions, instead having to rely on estimates using Stevenson's or (later) Shield's simple formulae, amplified by anecdotal wave height records, perhaps as quoted by Shield[25].

Critical to understanding of many of the early failures is the ability (or generally the inability) to anticipate and predict processes of wave shoaling on natural seabeds or artificial mounds, and thus the generation of impulsive breaking. There were occasional glimpse of clarity, although often marred by misjudgements. Scott[11] rehearses discussions on "*waves of translation of the 1st order*" and *"waves of oscillation of the 2nd order* " which appear to be based on previous work by Airy[12] or Scott Russell[13] but are not identified with any clarity. Scott enumerates ten points on waves of oscillation, amongst which he draws out the important conclusion: "*When the foreshore causes a wave to break upon the wall, the destructive effect is greatest.*" Scott then suggests that "... *when waves can be reflected, they ought not to be broken ...when ... the wave must break, then the operation should be spread over the largest surface and over the longest time possible*." He continues: "*The first condition will be fulfilled by a vertical wall, and the second by a slope.*" Scott uses this to recommend that the seaward face of breakwaters should be " ... *built nearly vertical...*" rather than sloped. On sloping faces, he notes that "... *the following wave breaks by falling against the lower part of the advancing bank of water ... it knocks the feet from under the advancing wave.*"

So in 1858, Scott[11] had laid the foundations for understanding of shoaling and breaking, but the lack of any agreed taxonomy, or of any analytical or empirical framework, inhibited engineers from taking this improved understanding further. Of the three principal manuals by Stevenson[7], Vernon Harcourt[9] and Shield[10] only the latter is clearer on the processes of waves shoaling and breaking over an approach mound. Shield notes that waves at Peterhead reached H_{max} ~8m, L ~ 150m, in 13-15m, then starting to break at the -10m contour (not unreasonable for $H_b/h \approx 0.78$). Yet no empirical methods are suggested. Citing Russell[12] comments on depth-limiting wave heights, Stevenson derives $H=0.4d$ which might be a potentially dangerous version of $H_{sb} \leq 0.55d$.

3.2 Wave forces / pressures

In discussing wave loads, Russell[12] notes that " *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, ...be left rather to accident ...when he has to decide on a system for best opposing the force of the sea…*"

In terms of wave load predictions, the designer in this period could use Stevenson's measurements[14], or back-analysis from known failures, or successes. For the wave condition at Peterhead discussed above, Shield imagines this wave hitting a wall, rising to just under 8m, and estimates a 'hydrostatic pressure' equivalent to 78 kPa. He then contrasts this with a 'dynamical force' equivalent to 292 kPa. Shield's calculation methods are not clear, but approximations may be given by assuming breaking wave velocities approaching wave celerity, perhaps given by $c = gT/2\pi$. For a 15s wave, the velocity might be 20-25m/s. A stagnation pressure given by $p = \rho u^2/2$ might therefore reach p = 200 to 300 kPa. The 'hydrostatic' pressure might indeed be given by wave run-up to 8m.

3.3 Blockwork stability

It was appreciated during this period that the stability of individual blocks in a wall depended significantly on generating inter-block friction[8]. Despite the importance of wall stability, but perhaps not surprising given the lack of any prediction method for wave forces, there remained throughout this period no prediction method for blockwork stability. Recent work[15, 16, 17] has brought such a prediction method nearer, but not yet developed a robust prediction method.

4. Improvements in construction

The most crucial improvements in materials and construction were those that allowed the toe level of the wall to be lowered, thus reducing occurrence of impulsive wave loads, and increasing resistance to sliding by increasing the self-weight of the wall. Some of those improvements took time to develop, or depended on other advances.

Development of concrete blocks to replace hewn stone blocks, from ~1855, accelerated construction, and substantially increasing block interlock, but had to wait until cement production had been commercialised, mechanical concrete mixers developed to supply the volumes needed, and steam-powered cranes were available to lift the larger blocks.

In turn, use of helmet-divers from ~1840 allowed foundation blocks to be laid at lower depths, substantially assisted when those blocks were shaped for interlock, and could be lifted by steam cranes. The main work of levelling the foundation would have required mechanical grabs lifted by steam cranes.

4.1 Masonry walls

Given the lack of any analytical advances in the design of masonry walls as discussed above, it is difficult to identify any substantial improvements in blockwork walls over the study period, until the rubble fill between the outer skins was cemented, or was replaced by concrete blocks. Then the major improvement was when all of the wall (including granite facing pieces) was replaced by interlocking concrete blocks, from about 1860.

The other areas of debate in this period were on the advantages of battering the wall to a slope, generally disapproved, and the advantage of a parapet wall, or not. In the absence of any analytical methods to quantify the effects of a parapet wall on either overtopping or wave forces, the discussions were essentially qualitative.

4.2 Level of wall foundation

Through the discussions in ICE Proceedings reviewed in this research[1], there was considerable debate by ICE members as to the safe level to which a rubble mound should be brought before the wall foundation could be set. Whilst not directly relevant to blockwork walls, it is interesting to note that the Great Chest caissons at Tangier may have been founded at 6ft (≈2m) below LW[19].