# Old British Breakwaters

# How Have Engineering Developments Influenced Their Survival?

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# PREFACE

## History of the project

The genesis for the project arose during consultancy studies at Wallingford from the late 1980s onward, particularly for Alderney breakwater, Dublin Great South Wall, and most recently the outer harbour breakwaters at Dover, and may be summarised by the question: 'how did we get to where we are now?' That question was given particular weight by observations of breakwaters that have failed, *e.g*. Portpatrick and Port Logan[1], Greve de Lecq[2], Wick[3], and Skateraw[4]. The various Alderney projects led to papers in 1991, 1998, and 2009, then to a discussion at the Institution of Civil Engineers (ICE) through the 2010 Vernon-Harcourt lecture "*Historic British Breakwaters: what did we build between 1600 and 1900, and how well have they lasted?*" But still many questions remained, driving a desire to follow-up the topic in greater detail, and answer (some of) the implicit questions.

The formal research for this PhD at Edinburgh started part-time in 2012 when full-time employment limited research time available. Moving to part-time employment in 2017 should have allowed more time for research, but in 2017-2019 progress was substantially delayed by a major forensic analysis project, and part-way through that project by leaving HR Wallingford and setting up my own consultancy. The consequential 'interruption' of the thesis research was formalised for 18 months ending in spring 2019. Since then, research on this project has been more consistent, but has still to be fitted in around consultancy projects.

## Input by others

In any project of this duration, especially in the absence of any external funding, I have had to take advantage of resources as they came available. To start with, I inherited archives from the BloCSnet project funded by EPSRC as a 'networking' project. This project therefore had no dedicated staff funding, but exploratory work by project participants was supported by short-term students at HR Wallingford and Queen's University Belfast (QUB) exploring issues behind blockwork wall failure. Work by Gerald Muller and co-workers at QUB and later at Southampton University then extended that with laboratory experiments to explore wave effects on and between blockwork in walls.

I have also used material that I developed during consultancy projects (and later) at HR Wallingford, particularly on Alderney and Dover. The 'orphan breakwater' project at HR Wallingford discussed in Chapter 7 was substantially assisted by interns, Nick Hanousek and Adrian Pearson. Nick and Adrian conducted the 'dry build' and 'wet flume' tests as described in Chapter 7. Adrian Pearson also developed a spreadsheet model of wall stability reported in Pearson & Allsop (2017), and Adrian assisted me in reporting the tests in Allsop *et al* (2017).

## Declaration

I, Nigel William Henry Allsop confirm that the work presented in this thesis is my own, subject to the qualifications for data searching during the BloCSnet project and the model testing in Chapter 7 described above.

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[1] Galloway, South-West Scotland;
[2] Jersey, Channel Islands;
[3] North-East Scotland;
[4] East Lothian, East coast Scotland.

# ABSTRACT

This research analysed UK harbour breakwaters constructed between 1663 to 1910. It includes a breakwater built in Tangier from 1663, to Dover outer harbour completed in 1910. The period of main interest is 1840-1900, and the structures studied have been narrowed to vertical (or nearly vertical) walls on rubble foundations. The principal research activity has been to distil from historical literature the key reasons for past failures and successes. In doing so, the research has reviewed the construction and performance of breakwaters protecting a number of harbours of refuge, discussed in a paper in ICE Engineering History.

Other results of the historical research were: generic guidance; geographic data on breakwaters around the UK; details of their design, construction, and performance, derived from historical manuals, reviews of historical structures, and professional papers and discussions. These were followed by stability analysis using empirical formulae to determine factors of safety for breakwaters at Wick (failed), Alderney (partly failed), and Dover (successful). These have demonstrated why the outer end of Wick breakwater failed catastrophically, and confirmed how robust are the outer breakwaters at Dover. The marginal stability results for Alderney confirm the reasons behind frequent damage leading to the abandonment of its outer end.

Historical records identify that designers of the period had no robust methods to predict waves, or their transformations. Even when engineers had decided what size or form of wave needed to be resisted, they did not then have reliable methods to calculate wave forces. As if that was not bad enough, the engineers of the day did not know how to make marine concrete, they had no steam power to move materials, nor the equipment to place foundation blocks underwater. So advances were by trial and error. After 1836, improvements were substantially assisted by ICE papers and the formal discussions at ICE. Probably the most important change was to lower the rubble mound foundation for the masonry wall. The further underwater that this junction could be placed, the more stable (and thus long-lasting) the structures became. Achieving this in practice however required new materials, construction tools, and methods. During the latter period, there were substantial developments: of cement to make concrete; of steam power to lift and move heavy items; concrete mixers to make concrete in bulk; and diving equipment to allow underwater working.

A very curious omission is that solutions to some of these problems were used at Tangier before 1680, but that breakwater was destroyed in the military withdrawal, and almost no trace was left in the technical literature.

A different route was where the rubble slope was extended so that its crest was well clear of the water, as at Plymouth, Holyhead, Portland, and (in part) Cherbourg, precursors to modern rubble mound breakwaters, but of little further interest to this project. The research described here has concentrated primarily on the improvements in knowledge, materials, and construction techniques that moved breakwater engineering from the frequent failures of Alderney breakwater, to the robust survival of Dover outer breakwaters.

As well as informing the onset and progress of damage under wave attack on unbonded blockwork walls, model studies present new data and prediction equations for wave transmission over failed blockwork breakwaters to predict the residuary protection available when such structures are damaged or collapse.

The thesis is in two main parts. Part 1 summarises evidence on design, construction and performance; develops concepts of failure and success; analyses selected breakwaters; and draws conclusions to the research questions. Part 2 reviews the historical literature to summarise what our predecessors were trying to construct; what materials and tools they had at their disposal; what hydrodynamic processes they understood (or did not); and to what extent (and how) did they share knowledge and understanding. The thesis is supported by Appendices summarising the historical timeline, identifying many UK breakwater locations, and some of the key engineers of the period.

# ACKNOWLEDGEMENTS

Various historical sources have been seminal to the historical research, particularly the Minutes of the Institution of Civil Engineers, including extensive discussions to ICE papers through the mid to late 1800s, the books by Thomas Stevenson, Leveson Vernon-Harcourt, William Davies, William Shield, work by HR Wallingford for Wick harbour, for the States of Guernsey and Alderney around 1990, for Dover Harbour Board around 2010. Research by Senicle on Dover and Smith on Portland are all gratefully acknowledged. Support and expert advice from Colin Partridge of the Henry Euler Memorial Trust on Alderney, and from volunteer staff at the Alderney Museum, are most gratefully acknowledged. 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 the historical ICE Proceedings without which this research would not have been possible.

My early workshop paper to Port & Harbour Research Institute (PHRI) in Japan drew on research at HR Wallingford supported by MAST I and II programmes of the European Union, and by the Construction Policy Directorate of the UK Department of Environment. Presentation and publication of that paper was supported by HR Wallingford and University of Sheffield, PHRI and JISTEC in Japan.

The ‘orphan breakwater’ tests in Chapter 7 were conducted at HR Wallingford by Adrian Pearson (Visiting Researcher from Edinburgh University) and Nick Hanousek (Industrial Trainee from Cardiff University), supervised and assisted by William Allsop. The experimental team are grateful for support from Equipment Sales (new wave paddle) and Instrument Support (wave measurement and profiler equipment), Paul Tong (rock sorting) and Clive Rayfield and team for assistance with the flume. Dr Stephen Richardson acted as Project Director for HR Wallingford. Reports were written by William Allsop using section drafts by Adrian Pearson and Nick Hanousek. Those studies were funded in part by the Institution of Civil Engineers Research and Development Enabling Fund (ICE R&D) as project 1315. Additional support for the testing was given by HR Wallingford.

Also at HR Wallingford, I would like to record my gratitude for access to background and case study information, including old reports on wave conditions at Wick, Alderney and Dover. Dover Harbour Board, States of Guernsey and States of Alderney, States of Jersey are particularly thanked for access to client reports and supporting information and photographs.

I am very grateful to University of Edinburgh for facilitating my PhD research, especially their generosity in supporting my part-time student registration. Support and guidance from Professors Tom Bruce and David Ingram (Edinburgh) are particularly and gratefully acknowledged, together with early advice from Professors Roland Paxton at Heriot-Watt University and Trevor Whittaker at Queen’s University Belfast.

I am also grateful for support and useful tips from coastal engineers far and wide particularly at and through the Coastal Engineering conferences (ICCE) and ICE Breakwaters conferences, my fellow students at Edinburgh who tolerated my occasional appearances and offered encouragement, and work colleagues at HR Wallingford. Lastly I thank my wife Dr Jane Smallman who tolerated many years of occupation of the study, and joined cheerfully in expeditions to find ‘breakwaters that are no longer there’.

# GLOSSARY

| | |
|---|---|
| Battered | A steeply sloping wall, close to vertical, given by V:H, *e.g*. 5:1 |
| Booms | Stop-logs, stacked logs or planks set in chases to exclude water / waves |
| Brandering | Strips, bands, or fillets applied to the main framing; perhaps timber fenders |
| Breakwater | Structure generally functioning solely to protect moorings, quays etc., against waves, generally with no quay function *per se*, sometimes of low-crest |
| Cement | The bonding agent in concrete or mortar, made by firing clay and limestone / chalk. Often misused by non-engineer authors to mean concrete or mortar. |
| Chases | Grooves set in masonry, perhaps to hold stop-logs |
| Draught | Generally the depth of a vessel below water, but also used historically to mean a (draft) drawing |
| Droving | The process of making the surface of a stone block flat |
| Lee (side) | Side of a breakwater / mole / pier sheltered from waves |
| Mole | Generally a breakwater or solid pier, often with a quay wall on the lee side |
| Ovolo string course | A moulded or shaped horizontal course, usually with curved element(s) |
| Pawls | Bollards, mooring posts. |
| Pier | Often a composite of a breakwater and a quay for mooring ships |
| Pierre perdu | Rocks heaped loosely in the water to make a mound or foundation (as for a sea wall, or a breakwater |
| Quay | Part of a harbour against which ships are moored, on top of which cargoes are handled, having no wave protection function *per se* |
| Return period | Average recurrence interval in years between events of a given severity |
| Rickle | An informal stack or pile of stones, perhaps elongated |
| Rusticated | Rough-surfaced masonry blocks having bevelled or rebated edges producing pronounced joints |
| Scabbling | Shaping a stone to square / rectangular form |
| Stugged | Where stone is worked over with a pointed chisel (punch). Often a droved margin is worked around the margin |

# NOTATION

| | | |
|---|---|---|
| $A$, $a$ | Empirical coefficient | (-) |
| $A_c$ | Armour crest level relative to water level | (m) |
| $A(n)$ | Normalisation factor used to describe directional spreading | (-) |
| $\alpha$ | Slope angle, but can be angle of wave obliquity to contours | (°) |
| $a_e$ | Air content used by Partenscky in estimation of wave impact pressures | (-) |
| $B$, $b$ | Empirical coefficients | (-) |
| $B_b$ | Crest width of rubble mound berm | (m) |
| $B_c$ | Width of caisson | (m) |
| $B_{cw}$ | Width of crown wall | (m) |
| $B_{eq}$ | Effective width of mound / berm in front of the wall, at ½ mound height | (m) |
| $B_{wl}$ | Structure width at static water level | (m) |
| $B_t$ | Width of rubble mound at toe level | (m) |
| $B_{rel}$ | Parameter giving effect of toe berm on wave breaking | (-) |
| $B^*$ | Relative berm width, = $B_{eq}/L_p$ | (-) |
| $\beta$ | Angle of obliquity of wave attack, but see Note 3 | (°) |
| $C_r$ | Coefficient of wave reflection, reflected / incident wave height | (-) |
| $C_t$ | Coefficient of wave transmission, transmitted / incident wave height | (-) |
| $C^*$ | Relative reflection coefficient | (-) |
| $C_{Fh}$ | Wave impact force reduction factor | (-) |
| $C_t$ | Coefficient of wave transmission | (-) |
| $C_{\beta 2}$ | Battjes load reduction factor for wave obliquity | (-) |
| $D$ | Particle diameter | (m) |
| $d$ | Water depth over toe mound in front of wall | (m) |
| $d_{eff}$ | Effective water depth | (m) |
| $F_B$ | Buoyant up-thrust on a caisson or related element | (kN/m) |
| $F_h$ | Horizontal wave force on wall element, often taken as pulsating | (kN/m) |
| $F_{hImp}$ | Wave impact force, horizontal | (kN/m) |
| $F_{h1/250}$ | Mean of highest 1/250 horizontal wave forces | (kN/m) |
| $F_S$ | Factor of safety | (-) |
| $F_u$ | Up-lift force on caisson or crown wall element | (kN/m) |
| $f$ | Wave frequency | (1/s) |
| $f_m$ | Frequency of peak of wave energy, = $1/T_p$ | (1/s) |

| | | |
|---|---|---|
| $g$ | Gravitational acceleration | (m/s$^2$) |
| $H$ | Wave height, crest to trough | (m) |
| $H_b$ | Breaking wave height, usually taken as maximum breaking height $H_{maxb}$ | (m) |
| $H_{bc}$ | Critical wave height for transition between pulsating and impact | (m) |
| $H_d$ | Design wave height, may be set to $H_s$ or $H_{max}$ depending on method | (m) |
| $H_{max}$ | Maximum individual wave height in design case | (m) |
| $H_{m0}$ | Significant wave height from spectral analysis, defined $4.0\sqrt{m_0}$ | (m) |
| $H_s$ | Significant wave height | (m) |
| $H_s*$ | Relative wave height to toe berm depth, = $H_{si}/d$, or local depth, $H_{si}/h_s$ | (-) |
| $H_{so}$ | Representative significant wave height at an offshore or nearshore position where it is substantially un-affected by shallow water processes | (m) |
| $H_s$ | Significant wave height, average of highest one-third of wave heights | (m) |
| $H_{si}$ | Incident significant wave height at toe of structure, taking account of refraction, shoaling and depth-limited breaking | (m) |
| $H_{ss}$ | Significant wave height (shoaled) | (m) |
| $H_{sb}$ | Significant wave height (breaking / broken) | (m) |
| $H_{st}$ | Transmitted significant wave height | (m) |
| $h$ | Water depth, varies with water level | (m) |
| $h_s$, $d_s$ | Water depth at the structure | (m) |
| $h_b$ | Depth over the mound or berm | (m) |
| $h_b*$ | Relative berm depth, $h_b/h_s$ | (-) |
| $h*$ | Relative approach depth to wave height, = $h_s/H_{so}$ | (-) |
| $h_b$ | Height of berm above sea bed | (m) |
| $h_{br}$ | Water depth at point of breaking | (m) |
| $h_c$ | Height of rubble mound / core beneath caisson / wall | (m) |
| $h_f$ | Exposed height of caisson or crown wall over which wave pressures act | (m) |
| $h_s$ | Water depth at toe of structure | (m) |
| $I$ | Pressure impulse, taken here as $p.\Delta t$ | (kN.s/m$^2$) |
| $I_{impact}$ | Pressure impulse under impact event, taken here as $p_{impact}.\Delta t$ | (kN.s/m$^2$) |
| $k$ | Wave number = $2\pi/L$ | (-) |
| $k_b$ | Empirical factor for influence on breaking of relative berm length | (-) |
| $K_s$ | Coefficient of shoaling | (-) |
| $K_r$ | Coefficient of refraction | (-) |
| $L$ | Wave length, in the direction of propagation | (m) |

| | | |
|---|---|---|
| $L_c$ | Length of individual wall section | (m) |
| $L_{mo}$ | Offshore wave length of mean ($T_m$) period, usually calculated using deep water assumption | (m) |
| $L_o$ | Deep water or offshore wave length - $gT^2/2\pi$ | (m) |
| $L_{pi}$ | Inshore wave length of peak ($T_p$) period | (m) |
| $L_{po}$ | Offshore wave length of peak ($T_p$) period, using deep water assumption | (m) |
| $L_{ps}$ | Wave length of peak period in water depth at structure toe | (m) |
| MSL | Mean Sea Level | (m) |
| $m$ | Bed slope | (1: - ) |
| $m_{rel}$ | Effective bed slope | (1: - ) |
| $N_{wo}$ | Number of waves overtopping, as proportion or % of total incident | (-) |
| $N_z$ | Number of zero-crossing waves in a record = $T_R/T_m$ | (-) |
| $P_{av}$ | Average wave pressure | (kN/m$^2$) |
| $P_b$, $P_{b\%}$ | Proportion (or percentage) of breaking waves | |
| $P_i$, $P_{i\%}$ | Proportion (or percentage) of impact events | |
| $p$ | Wave pressure | (kN/m$^2$) |
| $p_{av}$ | Average wave pressure, usually averaged over vertical wall height $h_f$ | (kN/m$^2$) |
| $p_{dyn}$ | Dynamic or impact pressure, used by Partenscky | (kN/m$^2$) |
| $p_{impact}$ $p_{imp}$ | Wave impact pressure | (kN/m$^2$) |
| $p_1$, $p_2$, $p_3$, $p_u$ | Wave pressures acting at points on wall calculated by Goda's method | (kN/m$^2$) |
| $q$ | Mean overtopping discharge, per unit length of structure | (m$^3$/s.m |
| $Q*$ | Owen's dimensionless overtopping parameter | (-) |
| $R_c$ | Crest freeboard, height of crest above static water level | (m) |
| $R*$ | Dimensionless freeboard, $(R_c/H_{si})/(\sqrt{s}(2\pi)$ | (-) |
| $r$ | Roughness or run-up reduction coefficient relative to smooth slopes | (-) |
| $s_m$ or $s_{mo}$ | Steepness of mean wave periods, $s_{mo} = 2\pi H/gT_{mo}^2$ | (-) |
| $s_p$ or $s_{op}$ | Steepness of peak wave periods, $s_{op} = 2\pi H/gT_p^2$ | (-) |
| $T$ | Wave period | (s) |
| $T_m$ | Mean wave period | (s) |
| $T_{m-1,0}$ | Spectral wave period based on $m_{-1}$ and $m_0$ | (s) |
| $T_p$ | Peak wave period | (s) |
| $T_R$ | Length of wave record, duration of sea state | (s) |
| $t_d$ | Duration of wave pressure / load | (s) |
| $t_r$ | Rise time, usually of wave impact | (s) |

| | | |
|---|---|---|
| $v_c$ | Wave velocity, used by Blackmore & Hewson | (m/s) |
| $x, y, z$ | Orthogonal axes, distance along each axis | |
| $z$ | Level in water, usually above seabed | (m) |
| | | |
| $\alpha$ | Structure front slope angle to horizontal | (°) |
| $\alpha_1$, $\alpha_2$ | Coefficients in Goda's method to predict wave forces on caissons | (-) |
| $\beta$ | Angle of wave attack to breakwater alignment | (°) |
| $\delta_0$ | Additional run-up height on vertical wall, used in Sainflou's method | (m) |
| $\gamma$ | Wave breaker ratio, $H_{sb}/h_s$ | (-) |
| $\eta_0$, $\eta^*$ | Extreme (notional) run-up level, used in Goda's and related methods | (m) |
| $\rho$ | Mass density, usually of (fresh) water | (kg/m$^3$) |
| $\rho_w$ | Mass density of sea water | (kg/m$^3$) |
| $\rho_r$, $\rho_c$, $\rho_a$ | Mass density of rock, concrete, armour units | (kg/m$^3$) |
| $\varphi$ | Internal friction angle | (°) |
| $\delta$ | Wall friction angle | (°) |
| $\Delta$ (delta) | Reduced relative density, eg. $(\rho_r/\rho_w)-1$ | (-) |
| $\Delta t$ | Time increment, often used as rise time | (s) |
| $\lambda$ | Model / prototype scale ratio (Froude) | (-) |
| $\lambda_e$ | Aeration factor used by Blackmore & Hewson | (-) |
| | | |
| $\xi$ (xi) | Iribarren number or surf similarity parameter, $= tan\alpha/s^{1/2}$ | (-) |
| $\xi_m$, $\xi_p$ | Iribarren number calculated in terms of $s_m$ or $s_p$ | (-) |
| $\xi_{br}$ | Breaking parameter used by Calabrese | (-) |
| $\theta$ (theta) | Wave direction relative to principal wave direction $\theta_0$ | (°) |

NB 1 Most definitions are given in full in the Rock Manual (CIRIA, CUR, CETMEF, 2007), ISO21360 (ISO, 2004), or the PROVERBS book by Oumeraci *et al* (2001)

NB 2 Some subscripts are condition-sensitive, so subscript $b$ indicates breaking, but only when dealing with wave conditions, e.g. as in $H_{sb}$; otherwise $b$ may denote a berm or mound.

NB 3 A cautionary note on use of $\alpha$ and $\beta$ as wave angles. In 'modern' nomenclature (since the Rock Manual, and ISO 21650, wave obliquities have been defined by $\beta$ (in plan) whereas $\alpha$ has been used as (vertical) slope angle. Some of the methods discussed in this thesis however date from before those (generally European) agreements, so use 'old' definitions. It may therefore be important to check definitions for context.

# CONTENTS

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# 1 INTRODUCTION

## 1.1 Outline of the problem

The UK has various river or estuary ports, but until recently trade and security of many parts of the UK depended on coastal ports protected against wave action by breakwaters. Small trading harbours up to large 'roadstead' harbours, all have been protected from wave attack by such 'walls in the sea'. To be successful, the breakwater must:

- emerge from the sea sufficiently high enough to exclude substantial wave action from passing over the structure;
- extend to provide sufficient calm 'shadow' under frequent wave attack from all likely wave directions;
- be sufficiently robust to maintain its own integrity even under rarely occurring severe wave condition;
- be sufficiently anchored, embedded or otherwise founded to resist wave (and tidal) forces;
- not cause adverse effects to neighbouring coastlines that might (for instance) lead to erosion / scour, or unwanted accretion.

This project has been concerned with the types of breakwater that have been most common around the UK, their *modus operandi*, and how and why they failed, or survived. The research has concentrated primarily on breakwaters that are vertical, battered, or horizontally composite (as a wall on a mound), so has not paid significant attention to rubble mound breakwaters for which there is a substantial corpus of knowledge. To limit the extent of the project, and to cover a period in which knowledge and technology developed rapidly, but often without adequate integration, the research has concentrated from about 1750 up to 1910. A single notable example has been drawn to start the story from 1660.

The primary motivation for the research has been to identify the key technical influences that led to the success or failure of representative breakwaters, and therefore to reveal how previous designers succeeded (or failed) in their designs and construction. For many structures considered, some reconstruction of evidence has been needed, more for those breakwaters that have 'failed', wholly or in part. Analysis of historical records suggests three primary influences on stability vs. failure:

- Improved understanding of wave / structure hydrodynamics identified some of the influences of wave height (and period) and mound level, on near-structure wave transformations, and thence on the magnitude and occurrence of pulsating vs impulsive wave loads.
- Materials available for wall construction expanded from quarried rock and dressed stone to pre-cast concrete blocks with potential for inter-block (and inter-layer) load transfer. Cement mortars allowed improved durability in bedding and sealing between blocks. Mass concrete in bags allowed foundation layers to be constructed rapidly with less opportunity for part-construction damage.
- Advances in construction equipment such as steam cranes, locomotives and tugs substantially improved construction speed, safety and efficiencies, and allowed much larger (concrete) blocks to be used to form breakwater walls. Use of helmet divers substantially lowered wall toe foundation levels, reducing (or eliminating) the occurrence of impulsive loads.

There are however complicated (and sometimes circular) interactions between these three areas, where each influence (at least in part) the resulting breakwater stability / longevity.

Until very recently, hydrodynamic analysis of near-structure wave processes was qualitative and often confused, so did not support quantitative analysis of the effects of wave conditions and mound / wall toe level on the magnitude and occurrence of impulsive vs pulsating wave loads. Quantitative analysis of wave loads on walls started *circa* 1925, but were not robust until 1985 – 2012 or even later. Until then, analysis or design of solid or blockwork walls used methods based on qualitative experience at other sites without significant quantitative normalisation for wave exposure.

1 INTRODUCTION

The general absence of reliable hydraulic cements up to *circa* 1850 severely limited the ability of blockwork walls to transfer loads internally to resist wave loads. Until it was possible to produce pre-cast concrete blocks with potential to transfer tensile / shear loads the use of only dressed stone limited the ability to share or transfer loads between blocks / layers.

Improved construction equipment, particularly steam powered cranes, locomotives and tugs, has allowed construction to be quicker, safer, and more efficient (cheaper). Availability of diving bells and helmet divers significantly improved the ability to construct the mound / wall toe to lower levels, and this made a major contribution to reducing the occurrence and magnitude of impulsive loads.

The availability (and hence role) of these and other less prevalent influences was substantially improved by effective technology transfer, primarily through ICE meetings and discussions, often consolidated by production of practice manuals. Limitations of the inherent design methods did however slow up-take of good practice, allowing poor design practice to persist well after better solutions had been developed, but not well shared.

The main period of interest for this research centres on 1840-1910 when many of the breakwaters reviewed here were constructed. As however some of the key pre-cursors to improved knowledge, materials and techniques started earlier, the study formally starts at Tangier in 1663 then held by the British, and ends with the expansion of Dover harbour completed in 1910.

## 1.2 Research objectives

To summarise the key objectives of the research, this project was focussed on determining for the period of interest here (~1670-1910):

1) How did breakwaters of this period fail?
2) What were the main causes of those failures?
3) What protection (wave shelter) was afforded by failed breakwaters?
4) What changed over this period to reduce failures, and thus improve resilience?
5) How were improvements promulgated and adopted?

A single simplified question has been posed to try to encapsulate most of the weight of questions 1), 2), and 4): "*why did Alderney breakwater keep failing yet the breakwaters at Dover worked right from the start*?"

In the course of pursuing questions 4) and 5) above, the research has summarised and reviewed much of the relevant technical literature of the period (principally 1840-1910), and those reviews have been included in Part 2 of this thesis, see section 1.4 below for the layout.

## 1.3 Structure types, examples, and key taxonomy

A breakwater is a structure in the sea intended to reduce wave action behind it. Breakwaters can be formed as mounds or walls, occasionally by floating pontoons, or sometimes by pile-mounted skirts or screens. They are <u>not</u> shore-normal beach control structures which should be termed groynes or bastions.

Traditionally, many UK coastal towns or country estates constructed their own small harbours for trade and to shelter fishing boats. Such harbours were particularly needed on 'rocky' parts of the coastline where the inland topography hindered construction of roads or railways. In the expansion of construction around 1770-1880, many such harbours were protected against wave action by a breakwater to reduce wave agitation, thus protecting quays, cargo handling facilities, and storage areas. There are essentially two main types of breakwater.

**Rubble mound breakwaters** are formed by dumping a mound of rock in a heap on the seabed, armouring front and back faces against direct wave attack by larger rock, or special concrete armour units. The slopes of the mound vary from 1:1.333 at the steepest to shallow slopes, sometimes as slack as *e.g.* 1:5. Most rubble mounds are designed not to re-shape under wave action, minor armour movement being allowed under extreme conditions, say 1:100 year return period. A special sub-division is the 'berm breakwater' where the armour may be smaller relative to the waves and some re-shaping of the outer armour may be permitted under more frequent conditions, *e.g.* 1:10 year return period. These types of breakwater require large volumes of rock, so may be most economical where costs of quarrying and moving rock are moderate.

Historically, breakwater engineers have however needed to minimise the volumes of material required, so have often preferred to build a **vertical, or steeply battered, wall**. These walls may be formed in timber, but mostly in masonry, later concrete. The most commonly-built such walls use two blockwork walls filled between by uncemented rubble. Later versions filled between the outer walls with cemented fill, even 'liquid concrete'. The availability of concrete allowed masonry walls and internal fill to be replaced wholly by concrete blocks. The behaviour of these various forms of 'vertical' walls are the main focus of this research.

The main responses (or 'actions') to wave loads are illustrated in Figure 1.1, most of which are discussed later in Chapter 5 and applied to selected case studies in Chapter 6.

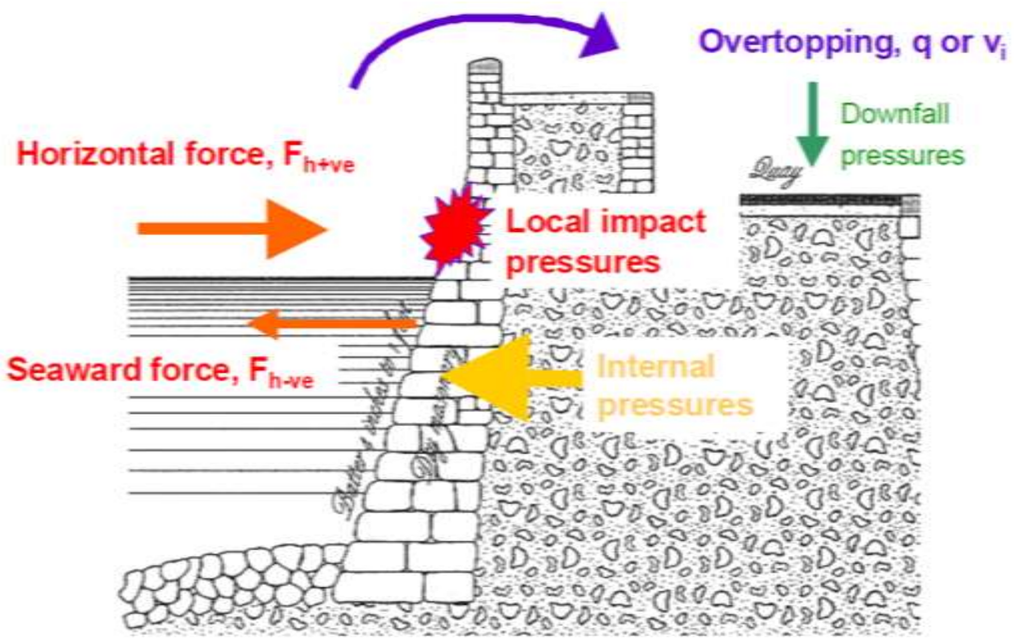


**Figure 1.1 Wave-structure interaction for a typical vertical wall breakwater**

## 1.4 Outline of the thesis

The thesis is in two Parts. Part 1 (Chapters 2-9) summarises evidence on design, construction and performance; develops concepts of failure and success; analyses selected breakwaters; and draws conclusions to the research question. Part 2 (Chapters 10-13) reviews the historical literature to summarise what our forefathers were trying to construct; what materials and tools they had at their disposal; what hydro-dynamic processes they understood (or didn't); and to what extent (and how) did they share knowledge and understanding.

The thesis is illuminated by two 'diversions'. The first was to identify some of the history of 'Harbours of Refuge' with particular reference to Alderney and Dover (also Cherbourg and Portland). A presentation was made to the Henry Euler Memorial Trust Seminar on Alderney in September 2019, and a paper developed from that is included in Appendix C

Appendix B tables test conditions for the research project described in Chapter 7, funded (in part) by the ICE R&D Enabling Fund on the stability and hydraulic performance of 'Orphan Breakwaters', elderly breakwaters no longer supported by significant maintenance or repair funds reported by further papers in Appendix C.

1 INTRODUCTION

Appendix D summarises the geographic review of UK breakwaters based substantially on documents compiled for ICE Historical Engineering, and by Graham and Willet. Appendix E presents biographies for many of the personae dramatis from the historical papers.

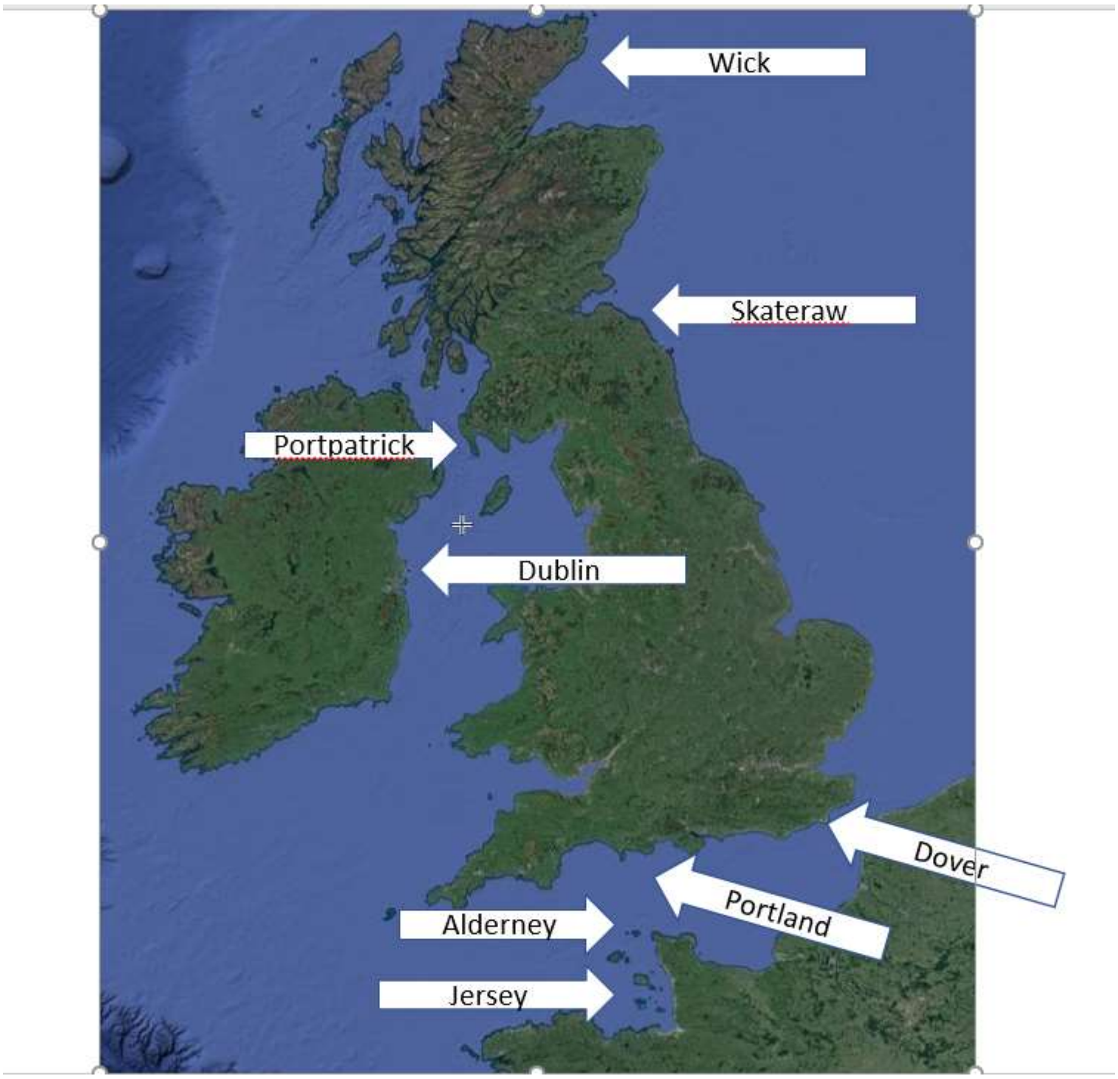


**Figure 1.2 Outline map indicating selected sites of interest**

**Part 1**

Following the Introduction of Chapter 1, Chapter 2 draws out the key examples of breakwaters relevant to the key research questions and discussed in the historical literature reviewed in Part 2. Chapter 3 continues by reviewing the main structure configurations, and the materials used. Chapter 4 then discusses the forms and methods of construction, highlighting particularly the construction plant developed during the main period of interest.

In preparation for case studies on safety analysis, Chapter 5 describes the key empirical analysis tools used to calculate wave loadings, and thus factors of safety. Then Chapter 6 presents the breakwater case studies to which the analysis methods in Chapter 5 have been applied. Wave conditions, summary structure geometry, wave loadings, and factors of stability are presented for Wick, Alderney and Dover, locations indicated in Figure 1.2.

Hydraulic model tests on idealised 'orphan' breakwater sections are summarised in Chapter 7 with guidance on the effects on wave transmission of failed breakwater sections.

Evidence from the case study calculations then highlight in Chapter 8 the key reasons for damage / failure to some of the more notable earlier structures, and the converse success of later breakwaters, with some lessons for analysis and future repairs / maintenance. Chapter 8 draws together the main lessons learnt from the historical review, analysis of failures and success, and the supporting model tests. Chapter 9 then summarises the key conclusions from the thesis.

**Part 2**

The historical review starts a general introduction to the **Historical Review** in Chapter 10, then with the most **General** literature in Chapter 11, material intended to be of generic use, particularly including three key manuals published in 1874, 1885, and 1895. Papers presented to meetings of the Institution of Civil Engineers (ICE) in this chapter may not necessarily have been intended as of general benefit, but the wide-ranging, intensive and extensive discussions to those papers are often major sources of background information and guidance – hence their inclusion in this part of the review.

To put into **Geographic** context, Chapter 12 has used the ICE Civil Engineering Heritage project and other reviews to summarise the extent and types of breakwaters around the UK. The coverage of this part of the review has been widened to include structure examples like lighthouses that identify particular materials or construction technologies relevant to the research.

From the first two relatively general reviews, the focus of Chapter 13 has then narrowed for breakwaters at fourteen **Specific Sites** which are discussed in more detail, from which are chosen the case studies covered in Part 1.

While each of the previous sections by their nature cover various aspects of construction methods, equipment, and materials, Chapter 14 identifies particular **Technologies** that have contributed to the successes of these structures.

The final section of this review in Chapter 15 is to identify some of the **Biographies** of key participants in design, construction and/or the ICE discussions. This section is not intended as original research, nor to be comprehensive, simply to give the reader some background to key participants.

An overall summary of people, places and technologies from the historical literature is given in the **Historical Timeline** table in Appendix A.

Appendix B tables the test conditions for a short research project described in Chapter 7, funded (in part) by the ICE R&D Enabling Fund on the stability and hydraulic performance of 'Orphan Breakwaters', elderly breakwaters no longer supported by significant maintenance or repair funds. Those hydraulic model studies exploring how idealised blockwork breakwater walls fail under wave attack, and measuring wave protection by the resulting debris mounds are reported by further papers by Allsop, Pearson & Bruce (2017, 2018) in Appendix C, and summarised in Chapter 7. Appendix D summarises the geographic review of UK breakwaters based substantially on documents compiled for ICE Historical Engineering, and by Graham and Willet. Appendix E presents biographies for many of the personae dramatis from the historical papers.

# 1 INTRODUCTION

# 2 EXISTING STRUCTURES, HISTORICAL REVIEW

## 2.1 Introduction

The material used in this chapter derives substantially from material reviewed in more detail in Chapter 13, and for the case study sites from Chapter 6. This chapter divides into four main parts, the first of which introduces British harbours (denominated as such by occupation), starting with some very early examples in section 2.2, the singular example of Tangier from about 1670 in section 2.3; and a listing of UK harbours constructed through the 17$^{th}$ to 19$^{th}$ centuries in section 2.4.

The second part (loosely termed Harbours of Refuge) starts in section 2.5 with an introduction to the main French and British harbours of refuge; starting with Cherbourg in section 2.6; followed by Alderney in section 2.7; the failed harbour at St Catherine's in section 2.8; and the more successful harbours at Portland in section 2.9 and Plymouth in section 2.10.

The third part describes briefly the failures of breakwaters at Wick, section 2.11 and Skateraw and Greve de Lecq in section 2.12.

The fourth substantive part of this chapter (section 2.13) is devoted solely to the outer harbour breakwaters at Dover, constructed from 1897.

The chapter finishes with some concluding remarks in section 2.14.

## 2.2 Early examples

Around the Mediterranean from around 2000 years ago, ancient breakwaters were often constructed of stone blocks, sometimes with concrete or cementitious infill. Roman engineers used underwater construction with timber forms (sometimes sunken ships), and filling with cement, pozzolana, and brick. A version of caisson construction was used by Herod the Great's engineers at Caesarea around 20 BC, where wooden forms were filled by concrete / mortar lowered in baskets into the forms; see Franco & Verdesi (1993).

Little evidence remains of Roman construction of breakwaters around the UK, although some foundations of quay walls dated to Roman times have been found in estuaries, particularly the Thames and Medway, where many quay walls were substantially formed by timber.

**Figure 2.1 The Cobb at Lyme Regis**
*From Bray & Tatham (1992), after Smiles (1874)*

On the open coast however, wave and tidal action is generally more aggressive, so few ancient breakwaters survive for long, nor do their records. Few details are available, but the construction of the Cobb at Lyme Regis (Figure 2.1) was probably before 1254 (Bull, 2015). The heart of the breakwater was formed by local Greensand stone retained behind oak piles, but that timber will have succumbed relatively quickly to rot, worm and abrasion. This type of construction was therefore gradually replaced by more formal masonry, perhaps secured by iron cramps and/or timber wedges.

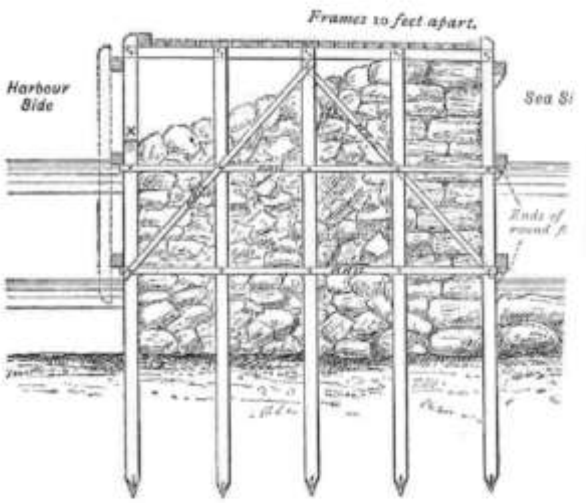


**Figure 2.2 Timber-framed breakwater**

*From Bray & Tatham (1992) after Shield (1895)*

Shield (1895) quoted by Bray & Tatham shows a 'typical timber-framed breakwater' in Figure 2.2, possibly termed crib-work.

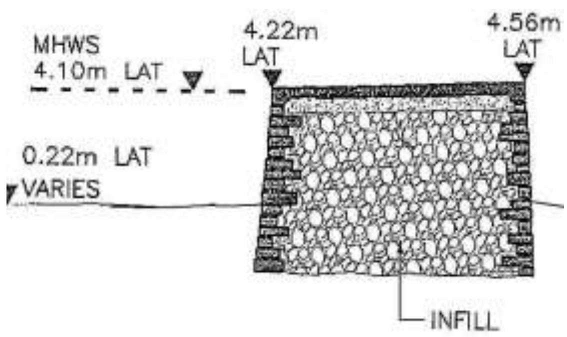


**Figure 2.3 Simplified section through Dublin Great South Wall**

At Dublin, work began in 1716 on a 'Mole' to protect the south side of the channel from Ringsend to Poolbeg, again using timber piles initially. [Possibly derived from the Italian word for a 'pier', the term 'Mole' generally means a breakwater, but perhaps one used as a quay on the lee side.] The early lengths of Dublin South Bank Wall provided only limited protection for shipping, so in 1753 it was extended as a stone wall using granite blocks from Dalkey to become the Great South Wall (Figure 2.3) out to Poolbeg Lighthouse, lit for the first time in September 1767. The Great South Wall (Figure 2.4) acted primarily as a training wall for the River Liffey, but along its outer end also as a breakwater.

**Figure 2.4 Inner face of Dublin Great South Wall**

*NB - The rock armour on the south face is a relatively recent addition.*

## 2.3 Tangier (early use of caisson construction)

One notable early example breakwater for which construction information is available is the the Greate Mole at Tangier built in 1663-1678 by British engineers described by Routh (1912). At the entrance to the Mediterranean (Figure 2.5), a fort supported by a harbour would have great strategic importance, as evidenced later by Spanish occupation of Ceuta and by the British of Gibraltar.

**Figure 2.5 Location of Tangier and Straits of Gibraltar**
*Courtesy Google Earth*

The origins of British occupation of the Fort at Tangier are more complicated than need to be discussed here (see Routh, 1912), but involve military stand-offs between the local war-lord (Khadir Ghaïlan), Portuguese troops, and varying threats from France or Spain. The principal 'enemy' varied in time. During the 1660s Ghaïlan (supported by Spain) clashed with the garrison at Tangier on various occasions, but was never able to seriously threaten the nascent port, strictly just an anchorage, and agreed several truces with Governors of Tangier.

Following a failed sally-out from the Tangier fort by a Portuguese garrison, Lord Sandwich sent British seamen ashore to man the defences. This effectively took control of the city, notionally to protect it against Ghaïlan, but perhaps also to ensure withdrawal by the Portuguese. Shortly after the British had occupied the city, Lord Sandwich surveyed the bay to find the best position for a new Mole (breakwater) to protect the anchorage, Tangier being subject to both Atlantic swell waves from the west, but also to strong easterly 'Levantine' winds.

Back in London a committee for Tangier was established which awarded a contract to build the Mole to Lord Rutherford, Sir John Lawson, and Mr Hugh Cholmley at 17 shilling/$m^3$. Cholmley (resident engineer 1663-1674) had previous experience of construction of a pier at Whitby. The form of construction was essentially a rubble mound brought up to low water, then surmounted by a blockwork wall. Pepys diaries recall for 12th January 1663: "*So I went to the Committee, where we spent all this night attending to Sir J. Lawson's description of Tangier and the place of the Mole of which he brought a very pretty draught*."[5] Later for 5th of

[5] Assumed here to mean a drawing.

February, Pepys writes "… *I to my Lord Sandwich and there staid, there being a Committee to sit upon the contract for the* (Tangier) *Mole, which I dare say none of us that were there understood, but yet they agreed of things as Mr Cholmely and Sir J. Lawson demanded, who are the undertakers, and so I left them to go on to agree, for I understood it not*." This level of contract negotiations continued, for on 16 February 1663, Pepys again wrote: " ... *at the Solicitor General's I found Mr Cholmely and Creed reading to him the agreement for him to put into form about the contract for the Mole at Tangier, which is done at 13 shilling the Cubical yard, though upon my conscience not one of the Committee, besides the parties concerned, do understand what they do therein, whether they give too much or too little*."

Construction operations started in June 1663, with the foundation started in August 1663. The construction targets were 15,300m$^3$ by end June 1664, then yearly 23,000m$^3$. The Mole was to be kept in repair for 5 years from completion at £6,000 / annum. Progress was however very slow, not least because payments were substantially delayed. Only 8,000m$^3$ had been placed by January 1665. The contract price was increased to 22.4 shillings/m$^3$ (31% increase), and progress improved such that a battery of guns could be placed on the Mole. Supervision by Major Taylor and Mr Cholmley (assisted by Henry Shere) was however interfered with by Colonel Norwood (Lieutenant General in 1666) "*intermeddling in the worke of the Mole*" who favoured onward progress to intermittent strengthening and consolidation. Despite the interference, progress on the Mole reached 350m[6] by August 1668.

In August 1669, Sir Hugh Cholmley was appointed Surveyor General, and the contract was now conducted by Government servants to a model approved by Christopher Wren and Jonas Moore. By April 1670 when Cholmley returned from England, the Mole had suffered serious damage in two winters, and doubts had been raised as to the wisdom of the current / intended construction. Henry Shere (Mr Cholmely's assistant) espoused a technique used at Genoa in which large wooden chests (caissons) filled with stones and cementitious material were sunk onto a foundation mound of stones and rubble. This approach, discussed at Tangier with visiting Genoese engineers in 1663, had been rejected by Cholmley as too difficult to make and install the caissons in a 3m tide and the exposure of Tangier Bay. Cholmley's preferred construction used a rubble mound up to low water as a foundation, surmounted by "*great stones cemented with lime and tarrace*[7] *and cramped with lead and iron*".

The extent of the Mole was surveyed on 19 April 1670 at 370 m long. Cholmley advised that only 70-90m further were needed, at 18m / annum, with a 55m return (to ESE?) to defend against easterly seas driven by the Levantine winds. Cholmley proposed to continue the current construction, but failed to convince the Governor, Lord Middleton, who backed Henry Shere's caisson suggestion.

Eventually, Cholmley agreed to try Shere's plan, and an attempt was made in September 1670 to place a first caisson. After significant difficulties, it was installed, although there was a suggestion that it was placed back-to-front. A second caisson was installed by early 1671. Progress continued through 1671-1674, but the winter of 1674/75 caused more damage to the Mole. Cholmely was much frustrated by restrictions and lack of payments from the Admiralty. In June 1675 he submitted a proposal for £30,000 a year so that he could repair and extend the Mole to 460m with a 90m SE return. He offered to return payments if the breakwater failed inspection, and to maintain it for £2,000 a year.

Cholmley's offer to continue the work was rejected in favour of a counter-proposal by Henry Shere who offered to repair and extend the breakwater using '*great upright chests*' (Figure 2.6) at £10,000 less than Sir Hugh's proposal. Henry Shere took the contract over in June 1676.

Shere placed his first caisson in September 1676 despite a strong east wind, and rough sea, and a second was launched in October 1676. Further were towed out from England and placed in summer 1677, each sunk into

[6] I am assuming here that the length given is of the wall. The mound will have extended further underwater.
[7] Tarras or tarrace was a local (slightly pozzolanic) material.

place filled by stone bound with the local mortar, example shown in Figure 2.6. By 22 October 1677, Shere had placed 11 chests, and the Mole reached 418m by November 1677. The caissons were so large that they were named as if a ship, including: Anglesey, Peterburgh, Craven, Coventry, Charles, York, Old Chest.

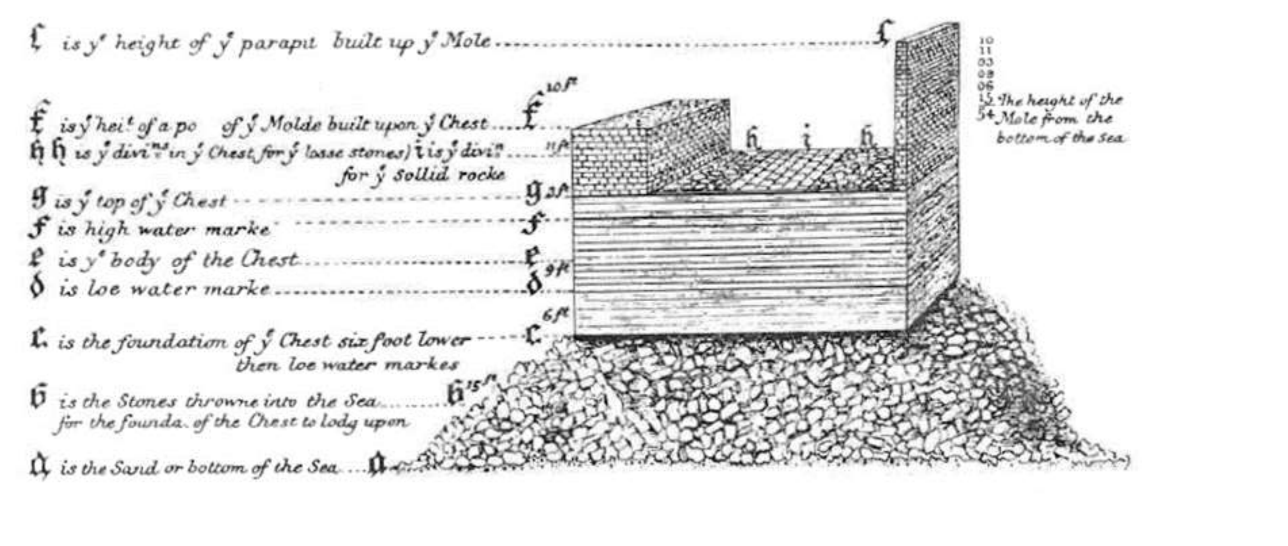


**Figure 2.6 The Great Chest constructed by Mr Shere, June 1677**
*After Routh (1912)*

Progress was now less subject to damage, but as before, Shere was troubled by payment delays, and was warned by Pepys not to buy materials in advance lest he be unable to pay his workforce. But that wasn't the only problem as the construction (and its quarries) were attacked in 1678-80 by the Moors who then laid siege to the town, diverting staff and materials to the fortifications. Peace was concluded in 1680. By February 1681 abandoning Tangier was threatened, including destroying the Mole to stop it falling into the hands of (prospective) enemies. Demolishing the harbour was finally agreed in an official report in 1683. Contributing to the recommendation to destroy the harbour were:

1. Depths in the harbour had been reduced by sand washed into the bay, and from attrition to the stone forming the Mole itself. It also appeared impossible to prevent siltation of the harbour;
2. The seabed sheltered by the Mole was full of rocks upon which (mooring or warping) cables were often snagged or cut;
3. The harbour was overlooked by sandhills from which the Moors could fire cannons;
4. The proposed breakwater return would make it difficult to leave the harbour under easterly winds;
5. Various Admirals preferred Gibraltar;
6. Seas would be expected to "*beat down the Mole*" if it was continued into deeper water;
7. The local water supply was insufficient for even two or three ships, and was "*bad and pernicious to men's health*".

Routh (1912) argues that many of these assertions were inaccurate, and that Shere told Pepys that "*he was able to answer them all*". Certainly, many independently minded naval captains favoured retaining Tangier harbour, but in vain as demolition of the breakwater continued through 1683 to February 1684.

It had been expected that Cholmley's early construction would be easier to demolish than Shere's later work. On the contrary, much more effort was required to demolish the wall over the inner sections, presumably because the slow-setting tarras and lime mortars had gained strength over the years. The demolition arisings were thrown into the harbour specifically to hinder any later use of the harbour area. Demolition of the Mole and the town took three months and required some 2000 men!

In demolishing the Mole at Tangier, the UK seems to have lost experience and understanding of this alternative type of breakwater construction as there are no signs of anyone later copying Shere's innovative approach. Most designers after 1690 reverted to the previous configuration of vertical or battered blockwork wall on top of a marginally-submerged rubble mound. The use of caissons in the UK may not have been repeated until the Mulberry harbour caissons used on the Normandy coast for D-day, see Institution of Civil Engineers (1948).

## 2.4 UK harbours in 17th to 19th centuries

Around the coast of the UK, particularly the coast of Scotland, the lack of railways and the generally poor state of road transport through the 17$^{th}$ and 18$^{th}$ centuries drove the need for harbours to support fishing and coal, and the wider marine trade. Many of these harbours are mentioned in Chapter 12, and are listed in the Historical Timeline in Appendix A.

Transport of coal drove many short-haul routes, particularly across the Firth of Forth, as soon as steam-powered vessels became available. Similarly, availability of steam trawlers, and their ability to follow the movement of (especially) herring increased the demand for new harbours on the Scottish and English east coasts. Between 1700 to 1800, construction and/or expansion at least 75% of the new / revised harbours noted in Chapter 12 appear to be linked to the transport of coal. On the east coast of Scotland, this expansion was followed by trade in lime for agriculture and for mortar production, then potatoes, and perhaps grain.

Around the UK, between 1800 to 1850, around 35 new breakwaters and/or harbours were started. A continuing expansion of (small) ports saw breakwaters or piers being constructed at:

> (1800-1810) St Austell, Fraserburg, Folkestone, Ardossan;
>
> (1810-1820) Aberdeen, Scarborough (started in 1736), Plymouth, Fortrose, Dun Laoghaire, Cullen, Gourdon, Portpatrick, Port Logan, Skateraw;
>
> (1820-1830) Donaghadee, Wick, Dunmore, Whitehaven, Stonehaven, Crail, Banff;
>
> (1830-1840) Seaham, Cove, Cockenzie (started in 1703), Reag;
>
> (1840-1850) Arbroath, Hynish, Kilrush, Brixham, Hartlepool, Holyhead, Alderney, St Catherines, Portland, Fraserburgh.

By now, the effect of Government policy on 'Harbours of Refuge' (see section 2.5) was being felt, particularly in the construction of new harbours at Alderney, St Catherine's, Dover, and Portland. Otherwise many new or expanded harbours were required to support expanded fishing fleets. A few new or improved harbours were required to support mail packets and related traffic, *e.g*. Portpatrick started 1820), Donaghadee (1821), and Holyhead (1847).

From 1850 to 1900, fewer new harbours were started *e.g*. Blyth, Newhaven, Torquay, Portsoy, Sunderland, and Peterhead, but most harbour construction was in expansion or revisions. Training walls to river ports were formalised by larger breakwaters protecting the entrances to the Rivers Liffey (Dublin Great South Wall), Tyne and Tees.

## 2.5 UK and French Harbours of Refuge, 19th century

Throughout much of the 19th century, Britain feared the growing strength of the French Navy. That fear was used by the UK government to justify construction of various coastal harbours[8]. The explicit threat from France abated with the defeat of Napoleon Bonaparte's armies at Waterloo in 1815, and his death in 1821. Fears of a French resurgence however, emphasised by strengthening of Cherbourg harbour, fuelled demands in the UK for 'Harbours of Refuge', debated at length throughout the 1840s, and discussed by Allsop (2020), Appendix C.

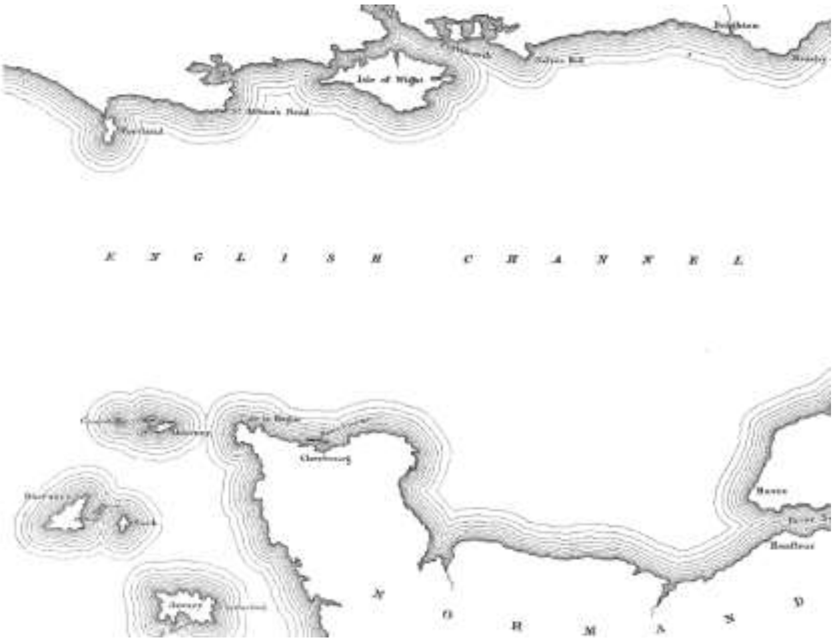


**Figure 2.7 Locations of Cherbourg, Alderney, Jersey, and Portland**
*After Vernon-Harcourt (1873)*

Harbours of Refuge were notionally conceived to provide shelter from storms for commercial vessels, including mail packets, fishing and general trade, see discussion by King-Noel (1848) in Chapter 11. Naval use of these proposed harbours was generally less explicit. At time of the design of these harbours (~1845 for most) large naval vessels were mostly powered by sail so it was difficult for a sailing vessel to leave harbour into an onshore wind without tugs. This limitation was understood in commercial operations. But even as the harbours were constructed, propulsion and form of vessels changed, with greater use of steam power, and iron or steel replacing wooden hulls - see Barnes (2014) and other papers discussing the changes of vessel power in Chapter 14.

In the UK, a sub-plot of the '*Harbours of Refuge*' debate was development of new harbours for the Royal Navy for defence. Less commonly discussed was their potential use for offence. At least three Royal Commissions debated at great length proposals for possible '*harbours of refuge*' at: Holyhead, Peterhead, Harwich, Dover, Seaford, Portland, Jersey and Alderney, see House of Commons (1847). The latter two harbours, close to the coast of France (see Figure 2.7), were potentially major defences against Cherbourg, the main French threat.

For reasons that appear incomprehensible to us now, see Davies (1983) and Allsop (2020), the UK government approved construction of breakwaters at both Alderney (section 2.7) and St Catherine's Jersey (section 2.8), starting in 1847. Construction at Portland started shortly after that, see section 2.9. The main harbour at Dover (section 2.13) was rather later.

[8] The decisions as to which harbours to be developed were somewhat incremental, and not always entirely clear, see especially the discussion on St Catherine's in section 2.8.

## 2.6 Cherbourg – the principal French 'threat'

The need for a harbour to protect La Manche against the British persuaded French military leaders to shelter the bay at Cherbourg (Figure 2.8) as a roadstead harbour.[9]

**Figure 2.8 Cherbourg on the Cotentin peninsular**
*Courtesy Google Earth*

At Cherbourg, three breakwaters were first mooted in 1665, but construction only commenced in June 1784 by the 4km long central breakwater, Figure 2.9. The design by de Cessart used timber cones, each 46m diameter at the seabed, 20m diameter at the top, and 20m high. The timber cones were then filled by stone over the lower part, and masonry-faced concrete on the upper part. Gaps between adjoining cones were later filled by rubble mounds. Figure 2.10 shows vessels depositing rock between the cones.

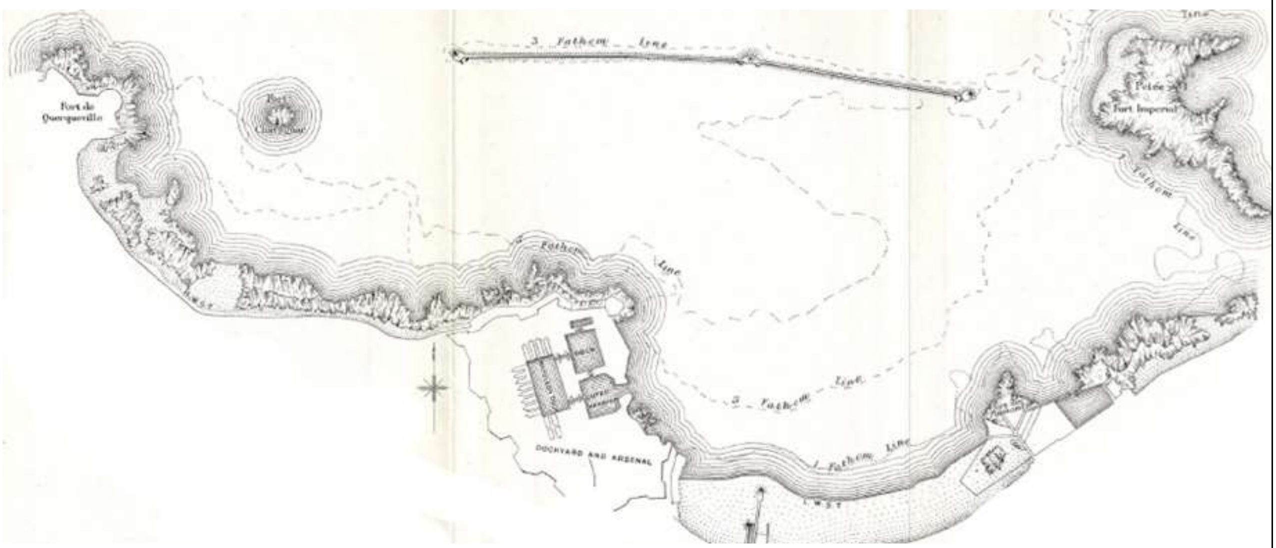


**Figure 2.9 Cherbourg harbour**
*After Vernon-Harcourt (1885)*

In 1802, Napoleon 1 restarted work on the central breakwater, reinforcing the centre to accommodate cannons. Large stones were used to raise and protect the crest in 1802 – 1803 for these gun batteries, but this rock was still moved by storms. In 1811 it was decided to take the battery foundations down to LW. Some 13,300m$^3$ of "*the largest stone procurable*" was placed in 1811[10].

[9] The term 'roadstead' implies a large anchorage that is partly sheltered from waves but is inherently less enclosed than a harbour. Cherbourg, Portland, and Dover were all conceived as 'roadstead harbours'
[10] Vernon-Harcourt (1885) neither identifies the rock size needed, nor that supplied.

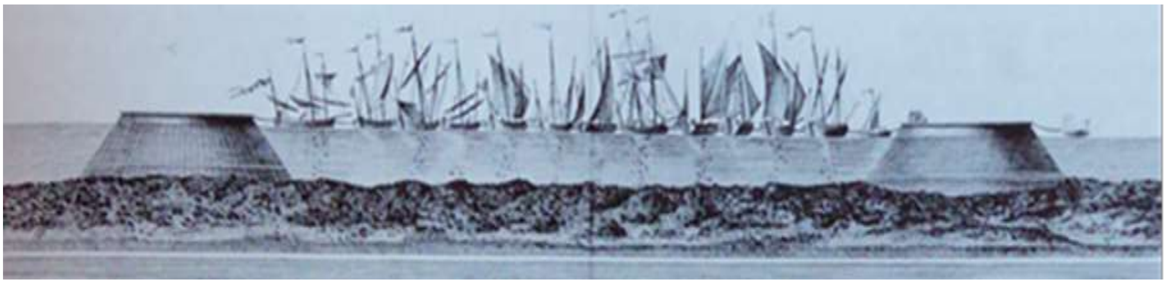

**Figure 2.10 Use of timber cones at Cherbourg**
*Courtesy Alderney Museum*

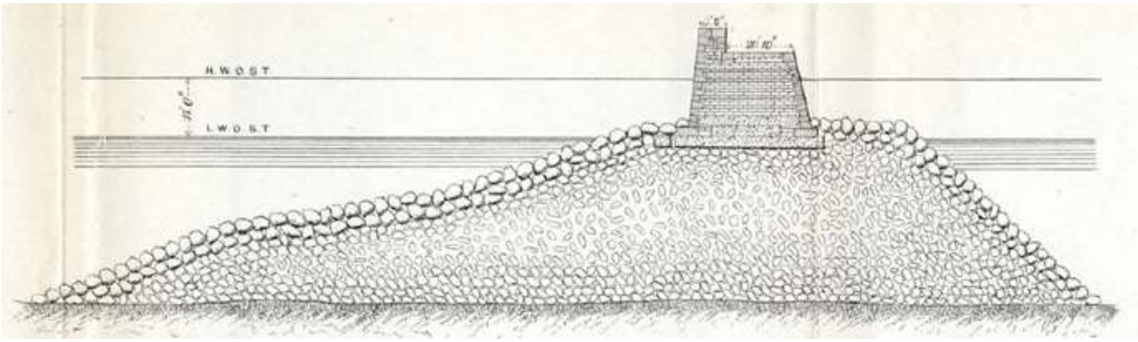


**Figure 2.11 Rubble mound at Cherbourg**
*From Vernon-Harcourt (1885)*

By 1813 the works were stopped, recommencing some 11 years later. Raising the breakwater crest above water re-started in 1830. Concrete blocks were cast in place on the rubble mound formed a toe / foundation, Figure 2.11. The lower slope was protected by large stones down to -5mLW at a slope of 1:5. The new superstructure suffered uneven settlement in the somewhat variable mound, so the final part was delayed "3-4 years to allow the mound to consolidate". The central breakwater superstructure was completed in 1846 under Louis Philippe I, and the pierhead forts in 1853. From 1846, work continued on the two side breakwaters Digue de Querqueville and Digue de l'Est, completed by 1895, enclosing the then largest harbour in the world.

The full potential for docking and shipbuilding were however never fully realised, apart from specialised submarine construction and maintenance (which continue). The harbour became a major transatlantic terminal from late 1800s, remains a significant ferry port, continues to accommodate a fishing fleet, and a base for submarine maintenance. So whilst the original 'cone' breakwaters were a substantial 'failure', the overall harbour with the later rubble mounds may be deemed a 'success', although at the cost of some recurrent maintenance expenditure.

## 2.7 Alderney

The island of Alderney is just to the west of Cherbourg in an area of high velocity flows where tides running up the Channel are compressed by the Cotentin peninsula giving the Swinge to the west, and the Alderney Race to the east. The western coast of Alderney is exposed directly to Atlantic storms. As a possible harbour of refuge, Alderney is well south of any coastal traffic along the south coast of England (Figure 2.7). Almost no civilian vessels would therefore require a refuge harbour on Alderney, and they might certainly prefer to shelter on the less wave-exposed east of the island. In the age of sail, a major naval tactic was to blockade one's enemy's fleet in its own harbour, which is why Cherbourg and Dover each have two entrances. But with the age of steam, fuelling a blockading fleet became complicated, so a convenient harbour from which to observe the enemy's harbour might be preferred.

**Figure 2.12 Alderney island**
*Courtesy Google Earth*

But why site this harbour on the most wave-exposed side of the island (Figure 2.12)? Again, the reason was military - to hide British warships from French telescopes on the Cotentin cliffs. But by locating the harbour on the wave-attacked west side, Admiralty planners effectively defined a troubled future for the harbour, and certainly for the breakwater.

Background to the selection of these sites is described by Vernon-Harcourt (1873) in *ICE Proceedings*, later in his book by Vernon-Harcourt (1885), and then by Davies (1983). Admiral Sir Edward Belcher explained to Vernon-Harcourt (1873) that he had been summoned in August 1842 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 early to allow estimates to be laid before Parliament. At Alderney, they found the tidal race across "*the mouth of the proposed harbour… would render it utterly impossible for any disabled vessel to get in…*"[11]. He suggested re-locating the harbour to Longy on the south-east side of the island[12]. Belcher's advice to the Admiralty was that a harbour at Longy would cost £1,500,000.

Even so, construction of the Admiralty breakwater at Alderney started in 1847 to a design by James Walker, 2nd ICE President, layout shown in Figure 2.13. The initial (section) design included a rubble mound to low water, surmounted by blockwork walls with rubble infill, (Figure 2.14). The mound was taken only up to a level where it was expected to be undisturbed by wave action. Stone for both mound and walls was quarried from Mannez quarry on the opposite side of Alderney.

Almost immediately the weakness of Walker's design became apparent. The foundation mound stone moved under wave attack and the breakwater wall was frequently breached. By 1849, sections of the extending rubble mound had been washed into the harbour, and considerable damage had been done to the walls. The design section was amended after 125m steepening the wall, masonry was set in Medina cement, and the seaward

[11] Probably Braye Bay
[12] But that would probably have made the tidal velocities even higher!

foundation was started lower. The foundation had been lowered after the first 46m as far as was practicable without divers. Having used end-tipping hitherto, the new lower mound level allowed use of hopper barges, but those required a construction harbour.

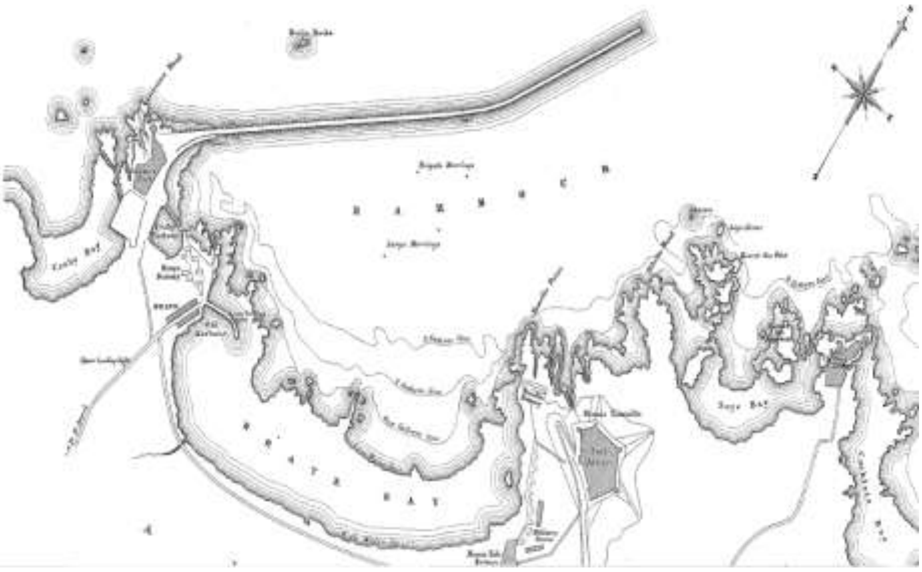

**Figure 2.13 Braye Bay and the Admiralty Breakwater**
*After Vernon-Harcourt (1873)*

In the revised design, the rubble mound was not disturbed lower than about -3.7m LWOST in the absence of the superstructure. Work to the revised design proceeded "*as soon as diving apparatus and the hopper barges were procured*". Construction continued to 823m by 1856. The design was then revised again, further lowering the wall foundation, now easier with the availability of divers. Construction of the outer section was nominally completed in 1864, giving a total length of 1430m.

But following repeated damage, including breaches, to the continuing construction, and substantial cost increases, Sir Francis Baring had summoned Sir Edward Belcher back to the Admiralty in 1852 to tell him that *"... the former Commission was still in force ... 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, designer of the breakwater, was also instructed to go *"... in order that he might be there in a gale.*" Walker and Belcher advised against an additional eastern arm, perhaps convinced that the concentration of tidal flows across the breakwater heads would scour their foundation mounds. Belcher concluded his contribution to the 1873 ICE 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.*"

Vernon-Harcourt (1873) noted that the idea of a further 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*".

Following breakwater completion in 1864, a storm in January 1865 forced two breaches of 15m and 40m through the superstructure. Another breach occurred in January 1866, a smaller one in February 1867, and another of 18m in January 1868. There were further breaches in December 1868, and in February and March 1869. By early January 1870, there remained two breaches of the superstructure along the outer part, and five other locations of damage. Sir John Hawkshaw (President ICE) and Col. Sir Andrew Clarke were requested by Board of Trade, who had reluctantly inherited the harbour, *"to visit Alderney and to report on the best measures for securing permanently*", either the whole (1740m) or the inner (870m) portion of the breakwater. Hawkshaw and Clarke noted instability of the mound and suggested removal of the upper 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 actions were taken.

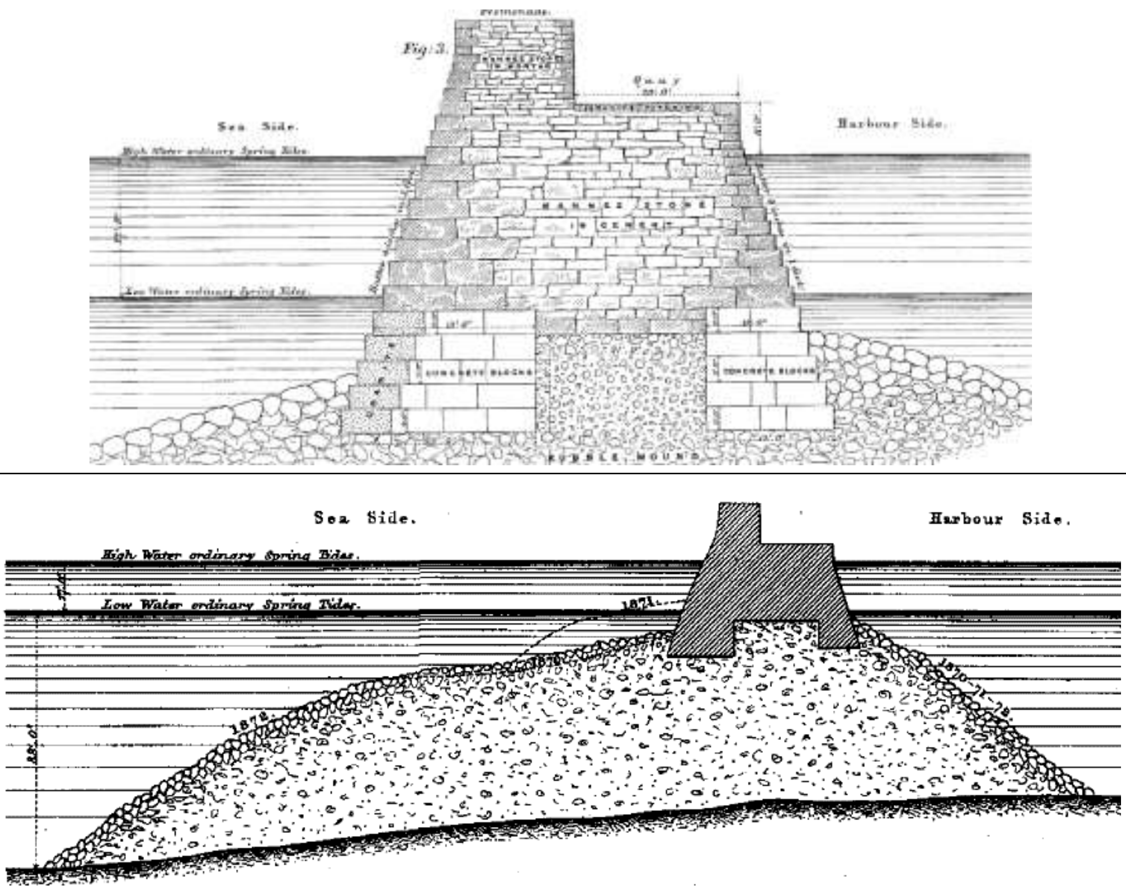


**Figure 2.14 Alderney breakwater sections**
*After Vernon-Harcourt (1873)*

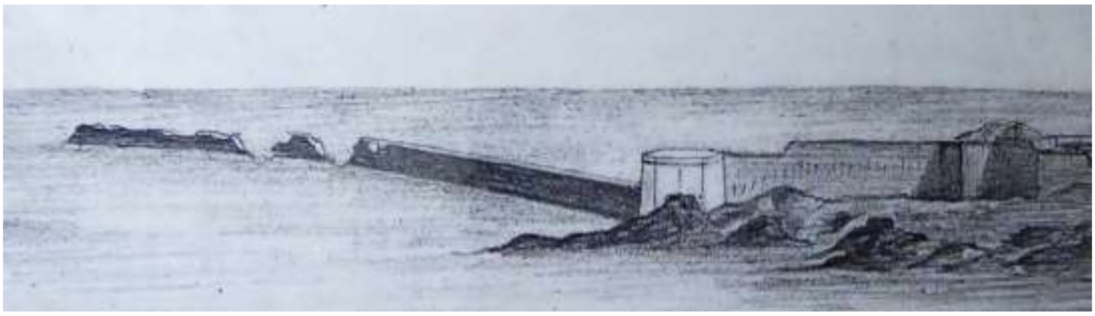

**Figure 2.15 Alderney breakwater in 1883, by Lt Farmar RA**
*Courtesy Alderney Museum*

The wall toe had been partially protected by stone dumped onto the foreshore. About 300,000 tons were tipped between 1864 and September 1871, after which it was decided to abandon the outer length. From 1873, repair and maintenance covered only the inner length of 870m. The outer portion was abandoned to the sea and the wall quickly collapsed (Figure 2.15) leaving a mound crest about -4mLW (Figure 2.16). 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 foreshore rock continued until 1964 except during the German occupation (1940-1945).

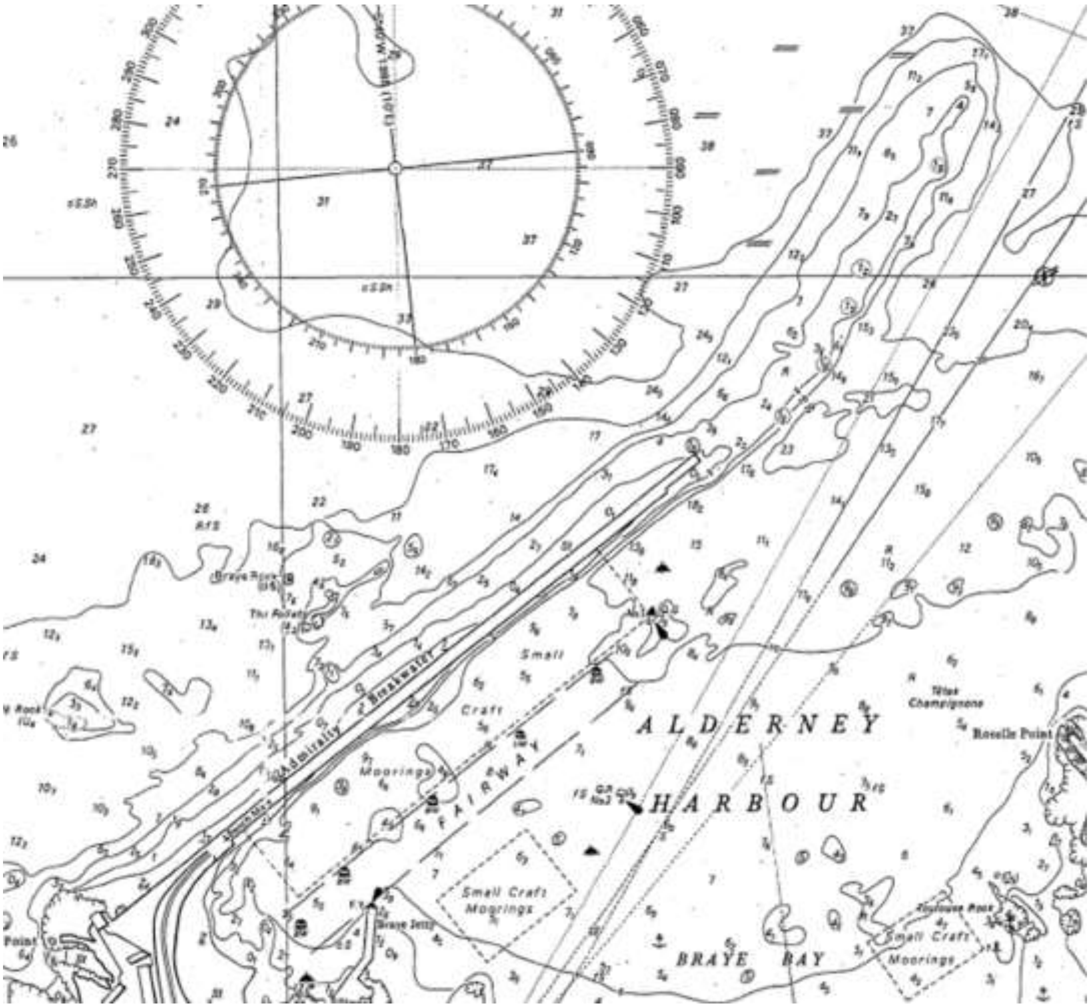


**Figure 2.16 Alderney breakwater showing the extended (submerged) mound, 1974**
*Courtesy States of Guernsey*

Waves at Alderney are frequently severe, (see wave condition tables in the case study in Chapter 6.) Depths off the breakwater generally exceed 15 to 20m and waves reach the breakwater with little reduction, with the 1:50 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 increased by waves shoaling over the mound, causing impulsive breaking. Storms usually persist for many hours, so the breakwater is exposed to the range of wave and water level combinations that allow waves to break directly against it.

In 1987 responsibility for the 870m long Alderney breakwater transferred to the States of Guernsey. Maintenance to 1990 cost around £500,000 per annum, excluding storm damage. That damage takes two main forms. Direct wave impact on the wall shakes the breakwater, and cracks mortar joints. Impact pressures force water into joints, and voids behind. Loose rock from the mound is thrown against the wall, abrading the wall by up to 1m. Over time, the typical size of rubble on the mound has reduced, and the process has generated considerable quantities of gravel and sand on Little Crabby and Platte Saline beaches to the south-west of the breakwater root at Grosnez Fort.

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 1:25 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 cracked the masonry facing, and a large cavity was formed in the wall which breached by an explosive failure audible around Braye. Other sections of the structure also suffered damage. An emergency procedure was in place, and repair work was underway within 10 days. Repair costs was estimated at £1.1 million. Studies by Coode & Partners and HR Wallingford explored potential solutions, see Allsop *et al* (1991). Alternative approaches to protecting this breakwater were later described by Sayers *et al* (1998), and recently by Jensen *et al* (2017).

Analysis of wave loads and factors of safety are given in Chapter 6, and by Allsop & Bruce (2020b) in Appendix C.

## 2.8 St Catherine's – a failed harbour of refuge

Two issues affect the utility of any 'harbour of refuge' on Jersey: whether that is a useful location at all. If so, where on Jersey might a harbour be useful? The plan by Davies (Figure 2.17) shows two breakwaters, both of which were started in 1847: St Catherine's to the north; and Archirondel to the south. The St Catherine's breakwater exists to this day, and has recently been refurbished (Hold, 2013). The Archirondel breakwater was planned to be 2.5 times longer, protecting the harbour from southerly and south-east waves, and from the northerly running tidal currents. But in July 1849 Walker instructed the contractor to divert effort to completing the northern breakwater, perhaps as the putative harbour started to silt up as the breakwater trapped sediment in the northerly drift. A stub of the Archirondel breakwater exists today Figure 2.18, probably in similar state to when it was abandoned.

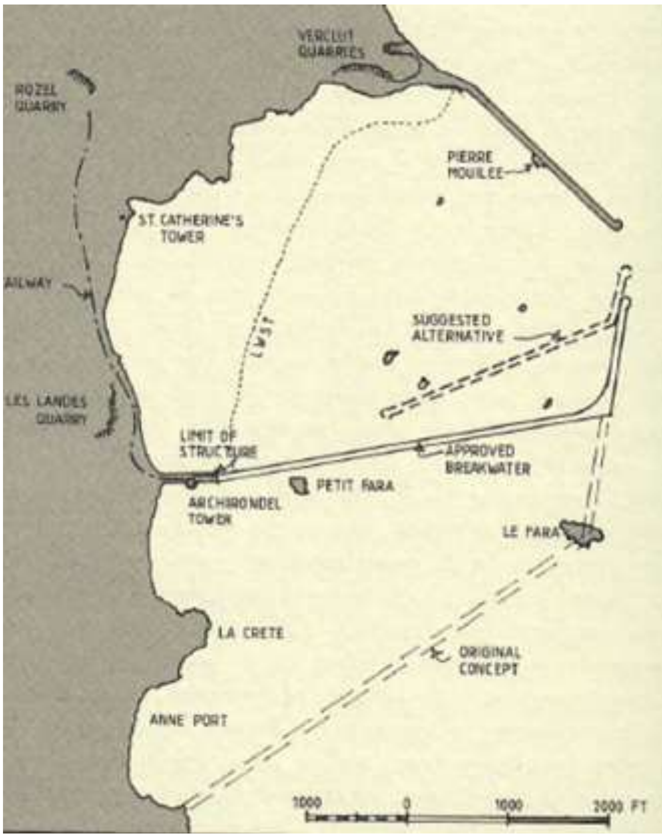


**Figure 2.17 Layout of the harbour at St Catherine's**
*After Davies (1983)*

Davies argues that siting a harbour of refuge on Jersey made no sense. This is an island of 12m tides. It is close to (but separate from) France to which it is nearly 'joined' by submerged rocks ESE to Coutances. Together with the substantial tidal flows between Jersey and France, these rocks significantly limit any trading vessel traffic along the east side of Jersey.

What about military use, even if not so declared? Again Davies (1983) rehearses the convoluted discussions. In 1831, Sir William Symonds favoured Bouley Bay on the north coast, although this had been countered by

**Figure 2.18 The Archirondel breakwater stub in 2014**
*Photo: the author*

Admiral (Rtd) Martin White (Jerseyman and navy surveyor) who "*unmistakably showed up the defects*" of that option. In early 1840s, Sir William Napier, Lieutenant-Governor of Guernsey, was requested by Whitehall "t*o 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, including Admiral Belcher, Colonel Cardew, Lieutenant-Colonel Colquhoun, supported by James Walker, Captain Sheringham (surveyor), some later involved in the Harbours Commission of 1844.

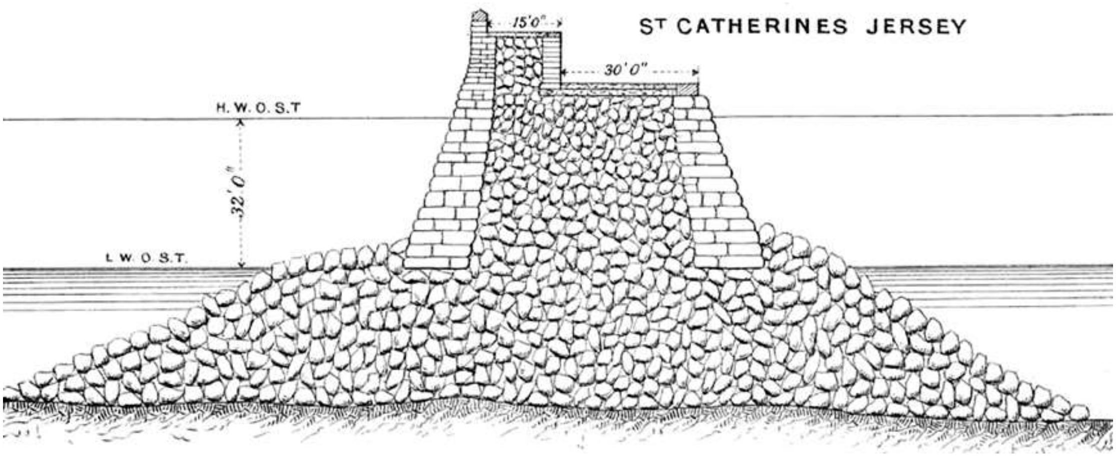


**Figure 2.19 Section of the breakwater at St Catherine's**
*After Vernon-Harcourt (1885)*

But by 1842, Government was ready to act. There were competing claims for Noirmont Point on the south-west coast of Jersey, or Bouley Bay towards the north-east corner. Davies notes that the national Harbours Commission of 1844, set up by 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*." It is likely that James Walker exercised his considerable 'networking' skills within Whitehall in favour of St Catherine's on the north-east coast[13].

Walker's breakwater design for St Catherine's (Figure 2.19) is very similar to that for Alderney, and its construction was relatively straightforward. Even if St Catherine's harbour could have been maintained, its utility would however have been severely limited by tidal conditions for which it could be accessed, and by sailing space between Jersey and France. The second threat was siltation, particularly sand driven by the northward-running tidal flows, depositing over slack water, made worse by cancellation of Archirondel breakwater.

By 1866, St Catherine's breakwater had been handed to the Board of Trade, Harbours and Lighthouses Department, whose Captain Bedford commented: "*it is anything but agreeable to take up and deal with the cast-off works of another Department – cast off too because they can find no use for them.*" There were various attempts to shift the problem, War Office, Home Office, back to Board of Trade, but the best option was to pass the problem to States of Jersey, despite their reluctance to take on an unwanted maintenance liability. The stand-off continued to February 1876 when the States passed a proposition to accept the breakwater, together with a 'dowry' of sufficient land to balance the anticipated maintenance liability. Negotiations with HM Receiver-General concluded in 1877 when it became the responsibility of the States of Jersey.

The failure of the St Catherine's harbour was primarily of utility, compounded by insufficient depth, and lack of interest of the States of Jersey, and the Admiralty. The breakwater itself has suffered little damage, most being confined to the outer end described by Hold (2009). Siltation of the harbour area was accelerated by constructing the breakwaters in the wrong sequence, capturing the sediment-laden northerly current, rather than deflecting it by Archirondel breakwater. No records exist of the changes of depth, but they must have been rapid to cause doubts on continuing construction beyond the first two years.

## 2.9 Portland – a successful harbour of refuge

Portland Harbour is another 'roadstead' harbour like Cherbourg, formed in the shelter of Chesil Beach and the Isle of Portland, Figure 2.20. The harbour was created initially by two breakwaters: the short inner or southern breakwater connected to the island; and a detached breakwater to the north-east with a 120m wide entrance. Construction began in July 1849, designed by J.M. Rendel, supervised by John Coode as resident engineer.

These breakwaters were simple rock-armoured rubble mounds with superstructures from low water. Portland stone was quarried from the island quarries by convicts, run-out onto the breakwaters over timber staging extending over the gap between inner and outer breakwaters, Vernon-Harcourt (1885). Timber piles (spaced about 10m apart and surmounted by creosoted cross-beams ~5.5m above HW) were founded on iron screws into the clay bed. Stone was dumped in ridges from the staging, "*the waves gradually levelled these ridges*". Large stones (3-7t) were dumped at an average of 500,000t per year from 1853 to 1860, reducing to 140,000t per year to 1866, giving a total of 5,800,000t. The outer (eastern) breakwater was then completed by two pierheads formed in masonry founded at -7.3mLW.

The harbour was declared complete by the Prince of Wales in 1872. As part of works against torpedo attack, two further breakwaters were added between 1893-1906. The present layout is shown in Figure 2.20.

[13] Walker's ICE obituary includes: "…he had, at least, as much skill 'in the engineering of men as of matter."

Portland Harbour was initially important for the Channel and later Home Fleets providing coaling and oiling depots. The harbour also became a base for the Admiralty's Underwater Weapons Establishment, and a factory and pier for torpedo testing. The harbour was active in both World Wars. The docks closed in 1959. The Naval Base continued for officer-training until RN operations at Portland ceased in 1995. The helicopter base closed in 1999. Portland Port was founded in 1996 as a private company to provide commercial and leisure uses, accommodating cruise ships and hosting sporting activities, including the sailing events in the 2012 Olympics.

Being constructed in the shelter of the Isle of Portland, wave attack on these breakwaters is relatively mild, certainly substantially less than at Alderney or Dover. Damage to the rubble mounds has been similarly moderate. The main naval need for the harbour lasted until 1959, so about 100 years of useful life. Portland may therefore be deemed a 'success'.

**Figure 2.20 Portland harbour**
*Courtesy Google Earth*

## 2.10 Plymouth

Plymouth Bay had provided the Admiralty with a major naval base since 1690. Sheltered as it was from waves from the west, the bay is however directly exposed to southerly storms, and to the prevailing south-westerlies. It therefore suffered very many shipwrecks, including 10 wrecks in a single day in 1804. In 1806, Lord St. Vincent commissioned John Rennie and Joseph Whidbey to explore protecting Plymouth Bay as a safe anchorage for the Channel Fleet. Rennie and Whidbey were assisted by Mr Hemans, the Master-Attendant of the Plymouth Dockyard. The three met in March 1806 and reported back in April recommending a fully detached breakwater line ring over Panther Rock, Shovel and St. Carlos Rock in the centre of the bay. An

**Figure 2.21 Plymouth harbour**
*Courtesy Google Earth*

Order in Council was issued by the Prince Regent in June 1811, and the foundation stone was laid on Shovel Rock in August 1812. Whidbey was appointed Acting Superintending Engineer. Stuart (1841) reports that the intended breakwater (Figure 2.21) would be 1700 yards long, of which the central section of 1000 yards would be straight and 250 yards[14] set back by (20°). The intended slopes were 1:3 seaward and 1:1.5 on the lee slope siting in around 10m depth at LW.

A new quarry was opened at Oreston in the Plym estuary, anticipating the need for some 2,000,000 tons of stone. The larger blocks (up to 7 ton) were hauled to the quayside in railway trucks pulled by horses, then shunted directly onto about a dozen sailing ships fitted with rails. Smaller boats (about 45) carried the rubble, using windlasses and tilting platforms so that it took only 50 minutes to moor, discharge the stone and set off to return. Initial progress appears to have been relatively rapid with stones visible at low water by March 1813, but severe storm damage in 1817 and 1824 prompted a change in the profile and height. Stuart reports that "*the spring-tide rose 7 foot higher than usual*" (2.1m), and "*a length of 796 yards of finished work was completely overturned*". Observing that the storm left material on the seaward face at 1:5, that slope was adopted for the repairs, with 1:2 on the leeward side. The crest was raised and widened to provide further shelter. From 1830, a foreshore was added increasing the breakwater width by 50 feet at the west and 30 feet at the east, requiring an additional 600,000 tons of stone. The upper slope and crest were paved in granite, see Figure 2.22.

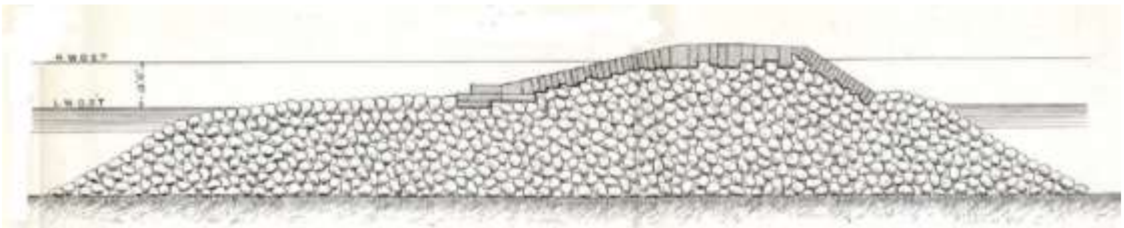

**Figure 2.22 Section of Plymouth breakwater**
*From Vernon-Harcourt (1885)*

By 1841, when the work is generally quoted as being complete, Stuart (1841) records use of 3,369,261 tons of stone, although later records suggest that 4,500,000 tons of stone had been used, and the cost was some 30% over the original budget, reaching more than 1,560m long. In discussion to the paper by Stuart (1841), Mr Rendel had analysed the quantity of stone used, and the volume occupied, from which he computed a porosity of 37%. At a time when interstices were often filled to improve solidity, he ascribes "*this great deficiency of solidity ... from the employment of an excess of large stone, or rather from a deficiency of small stone to fill the interstices between the large stones*"[15].

John Rennie died in 1821, and Joseph Whidbey retired around 1830. Design supervision was continued by George and Sir John Rennie.

## 2.11 Wick – mid-19th century fishing harbour

Wick Harbour in north-east Scotland (Figure 2.23) was a major fishing harbour in the 1700s and 1800s. Development by the British Fishery Society required further harbour expansion. Telford's (inner) harbour was completed in 1811, and an expanded (outer) harbour by James Bremner 1825-1834. Even so, by 1857 more capacity was needed so the British Fishery Society proposed a new breakwater (Figure 2.24). Plans, sections

[14] It is not clear where the missing 200 yards went!
[15] Rendel appears to believe that it was axiomatic how this was deleterious!

and specification for an expanded harbour were drawn up by D. & T. Stevenson in 1862. The design was supported by Sir John Coode and John Hawkshaw. The £62,000 loan was approved by A.M. Rendel as Engineer to Public Works Loan Commission. Construction began in April 1863 to a planned length of 460m.

**Figure 2.23 Location of Wick Bay, North-East Scotland**
*courtesy Google Earth*

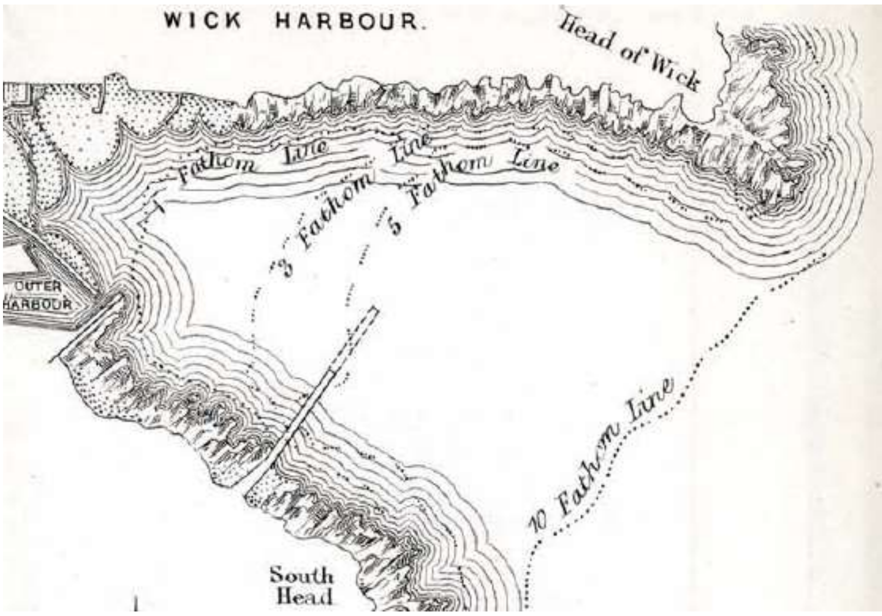


**Figure 2.24 Location of proposed breakwater**
*After Vernon-Harcourt (1885)*

The Stevenson design was a rubble mound to -5.5mLW following the Crane Rocks surmounted by block walls, filled between by rubble, with a superstructure width up to 16m (Figure 2.24). Rock for the rubble mound was hauled from South Head quarries by steam locomotives. Travelling gantries running on the staging then tipped stone onto the mound, possibly the first use of such gantries in Scotland. The seaward wall was formed as slice-work battered at 6:1. Below water, blocks were dry-jointed, but used Roman or Portland cement mortar

[16] above high water (HW). Paxton (2009) claims that the depth to which blocks were taken at -5.5mLW was 50% deeper than "*the accepted norm*" to avoid impulsive breaking over the mound. It is probable that "*the accepted norm*" was taken as 12ft below LW, although this has varied from 0ft in Walker's original design for Alderney, to the 24ft below used at that breakwaters 1862 roundhead. This is discussed further in Chapter 8.

By October 1867, the rubble had reached 326m, breakwater walls to 250m. By September 1868, the completed breakwater had reached 320m, but in October the outer 75m was demolished with the wall down to -4.6 m LW. Then in February 1870, the outer 116m was knocked over to -1.8 mLW (above wall foundation level) in a storm estimated at $H_{max} \approx 13$m, $H_s \approx 7$m.

In the summer of 1870, the outer 55m length was rebuilt (to 260m instead of the original 480m). The parapet was omitted, and the end was stepped. The top was rebuilt in concrete over coursed blockwork. At the vertical end, large concrete blocks were tied together by iron bars.

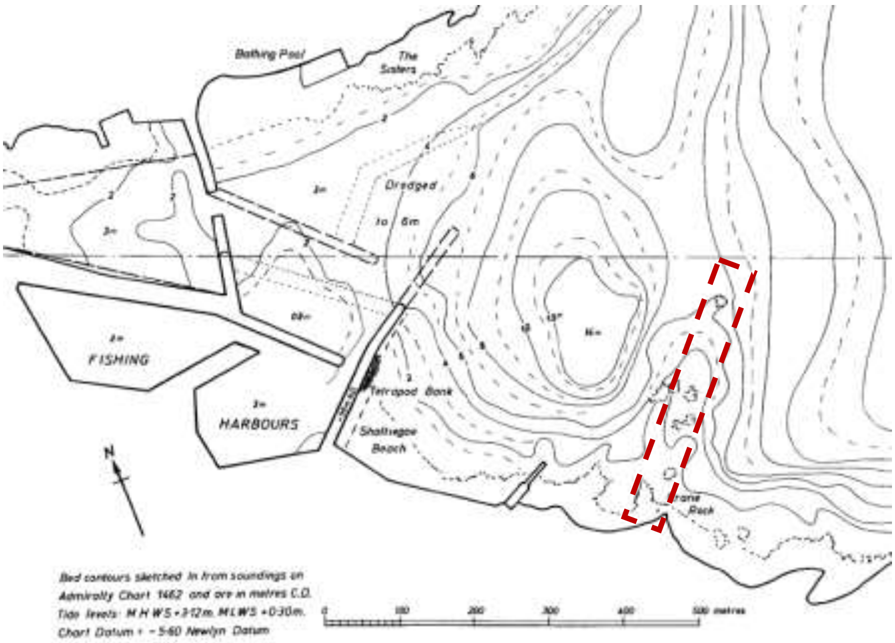


**Figure 2.25 Wick Bay bathymetry**
*After Hydraulics Research Station (1975). Position of Stevenson's breakwater is shown in the dashed box.*

In February 1870, the facing stones were shattered by an "unparalleled' sea, waves about 9m, spray about 60 m high. Then in December 1872, the replacement composite end (1372 t) was demolished down to -3mLW, being "*slewed round by successive strokes until removed*". The rubble foundation at -3mLW was undamaged. A further 46m "of solid structure set in cement" were then destroyed with water 8-9m deep passing over the parapet. After further damage in January 1877 when a 2642t end was destroyed in storms, Stevenson abandoned the project in August 1877.

[16] Stevenson (1874) fails to define 'Roman cement', so it is assumed to be portmanteau for any pozzolanic cementitious material.

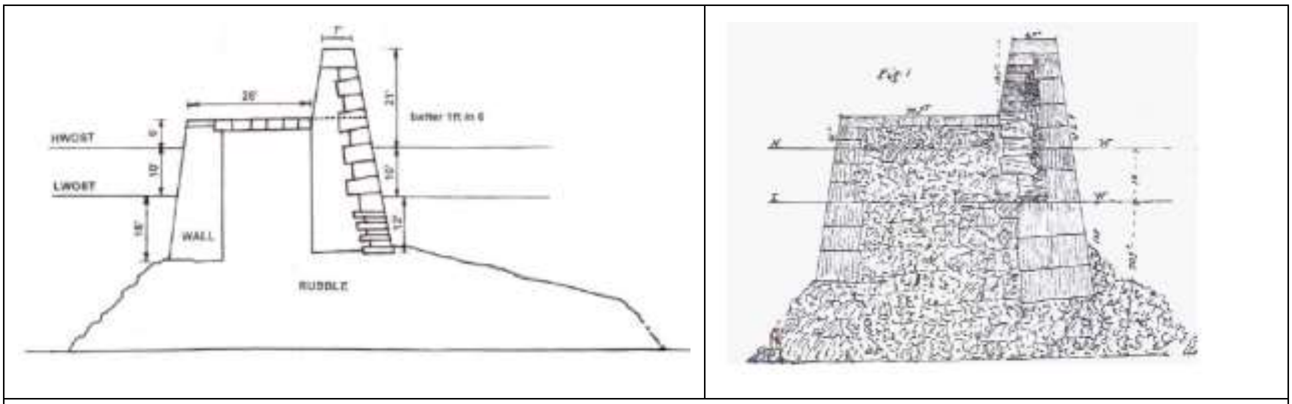


**Figure 2.26 Cross-sections of the intended pier**
*After Paxton (2009)*

The remains of the breakwater in Figure 2.27 show the wall at the root in relatively good condition. The remains of the head just showing above water are to the left in mid-distance (note the cranked angle near the shoreline in Figure 2.24). A small buoy marks the end of outer submerged mound.

The failure history by Paxton (2009) suggests both impulsive and non-impulsive wave loads. Additional loads will have been imparted by overtopping down-fall pressures, see Bruce *et al* (2001), Wolters *et al* (2005). The Safety Factor analysis in Chapter 6 uses wave conditions, including wave breaking to derive momentum-driven or pulsating loads, then extending to include impulsive loads.

**Figure 2.27 Remains of Stevenson's pier, May 2014**

## 2.12 Skateraw and Greve du Lecq – example small harbour failures

### 2.12.1 Skateraw - a collapsed breakwater

On the East Lothian coast south of Dunbar, an embayment at Skateraw (now just inland of the nuclear power station of Torness) had been identified as a potential landing place before 1791 (Figure 2.28). The harbour itself was built between 1799 and 1825 by two farmers, Brodie of Thorntonloch and Lee of Skateraw, who worked the local limestone quarries and kilns, shipping limestone to the Devon Iron Works (Clackmannanshire) and importing coal. A lime kiln stands beside the harbour site. It is probable that the harbour was also used to export potatoes.

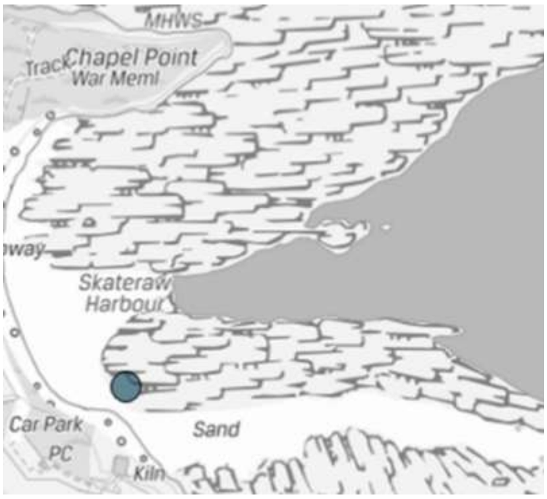

**Figure 2.28 Site of Skateraw harbour, 1825**
*Courtesy (Graham, 1971); Google Earth*

The original breakwater (just visible in Figure 2.29 and Figure 2.30) ran SE from the natural inlet with its entrance at the inlet's S corner. Graham (1971) gives the breakwater as about 85m long, about 9m wide, and returning at its SE end. Graham also argues that the NW end of the harbour was closed partially by a cross-pier about 30m long. The entrance between this pier-head and the end of the breakwater was about 8m wide.

**Figure 2.29 Collapsed breakwater and limekiln at Skateraw**
*Courtesy Google Earth*

The masonry of the breakwater is of long, relatively thin and well-dressed slabs, neatly shaped, but not apparently secured with cramps or keys. A large piece of natural rock, cut back on either side probably acted as a Pier. A mooring-ring set in its inner face indicate that it was a pier and not simply a foundation for a structure now washed away.

Little now survives of the harbour structures. It is surmised by Graham (1971) that the breakwater collapsed and the harbour was overwhelmed at some time between the survey dates of Ordnance Surveys of 1853 and 1892. The harbour area is now filled with sand and shingle in a tombolo in the shelter of the mound of the collapsed breakwater, concealing any evidence of the quay in front of the kiln, Figure 2.29.

The breakwater is reduced in its NW portion to a rickle of debris, in which, however, the line of the outer wall may be traced (Figure 2.30). Chapter 7 of this thesis will report physical model tests of simple blockwork walls which will illustrate the propensity of such walls to fail above the toe blocks. From this it may be judged that the visible blocks are probably 2-3 courses above the original foundation blocks.

**Figure 2.30 Collapsed breakwater at Skateraw (2014)**

### 2.12.2 Greve de Lecq – a partially-collapsed breakwater

The small harbour at Greve de Lecq is on the north coast of Jersey, Figure 2.31, to the west of the more sheltered harbour at St Catherine's. At Greve de Lecq the breakwater failure in 1884 was at part-length, Figure 2.32. An engineer's report after the failure (Hammond-Spencer, 1885) notes that: "*On 21 December the sea made a breach in the outer wall of the pier, which gave way under the violence of the north-westerly gales then prevailing, and more especially that of the hurricane in the early morning of Saturday 20th*."

**Figure 2.31 Site of Greve de Lecq on the north coast of Jersey**
*Courtesy Google Earth*

He continues: *"…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." "Looking at the quality of the work in the wall now standing, no one can be surprised that such holes as were known to exist should have been formed by the action of the waves and tide."*

Hammond-Spencer had previously reported on an earlier failure of the pierhead in 1879 when he wrote: "*It is not necessary to be an engineer in order to see that the quality of the masonry is altogether wretched and worthless, and that if once holes were formed in the face (thus affording a hole for the sea) the collapse of the wall would only be a question of time, longer or shorter according to the severity of the gales.*" He also criticised the design of the pier, arguing that the plan layout "*caused the force of waves to be concentrated on the section which collapsed*". He appears not to have approved of the outward curve of the pierhead.

**Figure 2.32 Failure at Greve du Lecq, Jersey, 1884**
*Courtesy Jersey Harbourmaster; and in 2014 (the author)*

His report recommended the rebuilding of the wall but went on to identify further problems. "*I consider the most important and difficult point to deal with is the lower portion of the masonry, at and about the level of low-water spring tides. This is the real weak point of the whole work, not only on account of the difficulties in working there but also of the exposure to which it is subjected.*" He notes that a close inspection at low tide revealed the state of the pier to be "*exceedingly bad, not to say alarming*", with holes extending almost to the centre of the interior of the pier, allowing the sea to suck out the infill.

And concluding, Hammond-Spencer writes: "*I am convinced that it is a mere question of time for the destruction of the entire length of the old wall which remains, from the east end of the existing breach up to the head which was rebuilt in 1879.*"

It is probably not therefore surprising that the remains of the collapsed breakwater wall at Greve de Lecq were never formally re-built, simply securing the shoreward end of the remaining wall, and cementing the collapsed material in-situ, probably at approximately its equilibrium level, see Figure 2.32.

## 2.13 Dover – the 'final' harbour of refuge

The Royal Commission of 1840 favoured a deep-water harbour in Dover Bay to enclose 450 acres (18.2km$^2$), cost £2,000,000. The 1844 Royal Commission re-considered whether a harbour of refuge was desirable here, requiring it to deliver, in order of precedence:

a) Ease of access for vessels "*requiring shelter from stress of weather*";

b) For armed vessels in event of hostilities, both offensive and defensive;

c) Should "*possess facilities for ensuring its defence*" against attack.

**Figure 2.33 Dover Harbour layout**
*Courtesy Google Earth*

Whilst this harbour was in theory to be for civilian vessels, military purposes were clear from the start. The 1844 Commission accepted the proposed site and general plan layout of the new outer harbour. A third Commission in 1845 considered plans by eight engineers for a harbour of some 520 acres (2.1km$^2$) out to 7 fathoms (12.8m). The outer breakwater was to be aligned with tidal flows to reduce siltation. The Commissioners reported in 1846 in favour of Mr Rendel's design. In comment, Vernon-Harcourt (1885) noted damage to sloping solutions at Cherbourg and Plymouth, and the lack of suitable stone at Dover. He also notes the shortage of experience in concrete. But given the chalk bottom, absence of local rock, "*and a moderate depth, the upright wall was the best system to adopt*".

The issue of siltation was again of significant concern, although this commission commented rather testily: '... *if liability to silt were deemed an objection, it would be idle to attempt such works on any part of our coasts'*. A contract was let in October 1847 for 244m of Admiralty Pier. Subsequent contracts in 1854 and 1857 covered further 305m, so that the work extending Admiralty Pier was essentially complete in 1871 to 640m from the shore.

Admiralty Pier forms the western breakwater to the harbour, and the extended end was constructed by 7-8ton concrete blocks with outer stone facings. The main wall was "*surmounted by a high parapet, overhanging* *considerably to the seaward"*. But on 1 January 1877 about 300m of this parapet at the outer end was swept away down to quay level. Wilson (1919) ascribes the blame to the curved overhang, although the slender nature of the up-stand wall, and absence of any tensile reinforcement must surely have contributed substantially. The damaged section rebuilt with a significantly thicker (about 3.3m) vertical face "*proved perfectly satisfactory*".[17]

This single pier did not however give any significant shelter from easterlies, and a contract was let by Dover Harbour Board in 1892 to Sir John Jackson to construct the Prince of Wales Pier to enclose the Western Harbour. The Prince of Wales Pier was extended to some 500m supervised by Coode, Son & Matthews. Then in late 1895, Coode was requested to prepare drawings to facilitate expansion to the full Admiralty Harbour (Figure 2.33) by:

a) Extension of Admiralty Pier by a further 610m;

b) A detached breakwater, the South Breakwater, of 1284m;

c) The Eastern Arm of 1012m.

This revised layout altered the length and overlap of the Admiralty Pier extension, and the position / width of the Eastern entrance, with the aims of improving accessibility to vessels, and reducing siltation. The Coode design was approved by the Admiralty, and a contract let in November 1897 to S Pearson & Son.

The new walls were formed almost entirely by concrete blocks (generally 2.3m wide and 1.8m high, depth from 2.4 to 4m to accommodate the 12:1 batter and ensure adequate bonding. Jointing was strengthened by half-height joggle joints, filled by 4:1 concrete rammed into canvas bags. At outer ends, tensile strength was increased by bull-headed rails turned down at the ends and let into chased channels / holes filled by 2:1 cement mortar.

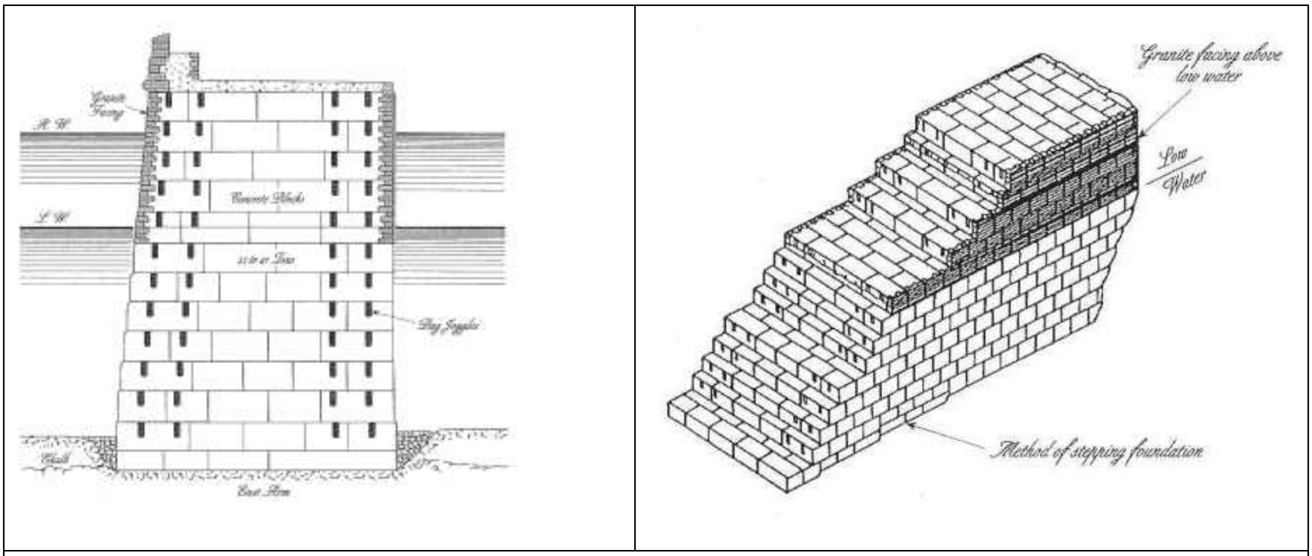


**Figure 2.34 Concrete block construction of Dover East Arm**
*After Allsop (2009); after Wilson (1919)*

For the foundation layers, underwater blocks were set by divers, placed tightly without mortar. Above the low water course (a band 1.8m high centred on LWOST) four courses were grouted by 2:1 Portland cement mortar. The Eastern Arm and Admiralty Pier Extension carried parapet walls, but such additional overtopping protection was not needed on (most of) the South Breakwater as mooring against its inside face was not

[17] A brief analysis of this failure is included in the case study calculations in Chapter 6.

envisaged. Mass concrete and granite pavers completed the crest. The parapet wall on the Admiralty Pier Extension reached 7.5m above HWOST (+13.3m LWOST).

The East Arm breakwater projects south for 900m. The section is similar to Admiralty Pier Extension, although the parapet wall was lower with the harbour cope at +8.8m LWOST), Figure 2.34. Foundation blocks for the East Arm were laid direct on the chalk or the chalk marl / flint matrix down to -16.2m LWOST. The East Arm was intended to provide berthing, so the harbour face was vertical with timber fenders, and an L-shaped head to shelter the inner face.

The South Breakwater (the Island Breakwater) runs 1284m parallel to the shoreline. Placement of blocks for this wall started short of the eastern end, allowing a later adjustment of the width of the eastern entrance guided by wave penetration and flows during construction. A curved section connected the eastern end to the main run of wall using curved blocks to maintain block tightness. No parapet wall was used along the main section of the South Breakwater, simply being added at the ends to provide shelter to buildings close to the roundheads.

To form the concrete blocks, cement (mostly from the Wouldham Company which Pearson had purchased) delivered by barge in 160t loads was derived from 'ordinary- and rotary-kiln' production. Wilson notes that the rotary-kiln cement was “*usually far quicker setting*”, so the two cement types were mixed. Concrete was mixed in two electric 'Messent' mixers of 1 $yd^3$ capacity. Output averaged 100$yd^3$ per mixer per day. Blocks were lifted after 7 days and stored for 3+ weeks. Two lifting holes ran through each block for the T-headed lewis bars. Blocks within the storage yard were moved by two 42t travelling Goliath cranes, then on stripped-down steam locomotives. Facing blocks included granite cast into the rest of the overall block. Granite was supplied from a Pearson-owned quarry in Cornwall, supplemented from Sweden, requiring special permission from the Admiralty.

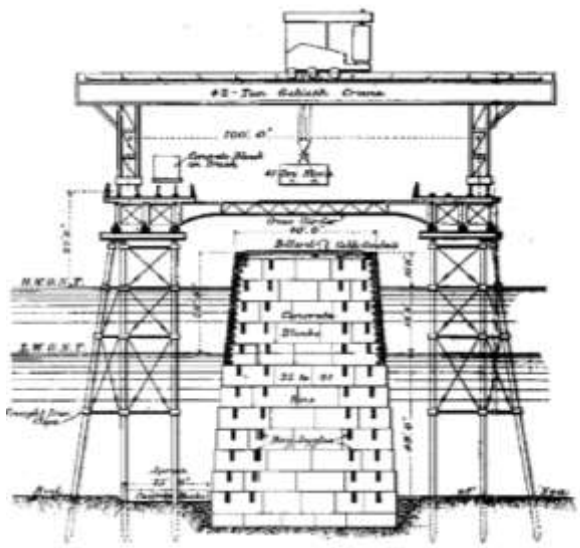

**Figure 2.35 Goliath crane on staging used for Dover South Breakwater**
*From Wilson (1919)*

Pearson eschewed the use of Titan block placing cranes that would run along the constructed works in favour of temporary staging above and beyond the works, supporting steam powered Goliath travelling cranes, Figure 2.35 and Figure 2.36. The rail level for these cranes was generally above +8.2m HWOST. Tasmanian Blue Gum piles were heavier than water, but Oregon pine required weighting by old iron rails to sink them. Staging piles were re-used as the work progressed, extracted by winch from a floating hulk. After use, piles were spliced to ensure availability of an undamaged head for driving.

Ahead of block placing, the seabed was prepared by excavating 1.5m of surface material, most by a 'Hone grab'. The final 0.3m of excavation was removed by four men using picks and shovels within a 35t diving bell which excavated a 4.6m wide strip across the running face, sufficient for two rows of blocks. The bell passed over each strip to give a coarse levelling, "*within a few inches*", and then a second pass for final levelling. Working under compressed air continued day and night in 3-hour shifts.

**Figure 2.36 Construction of South Breakwater using Goliath crane on staging**
*Courtesy Dover Harbour Board. Note additional 'sway' bracing relative to Figure 2.35*

Block-setting was supervised by two helmet-divers, blocks being placed hard against their neighbours. Significant effort was devoted to checking and regularising these courses to ensure an even base for the subsequent blocks. Bag joggles were placed by the divers, or from within the bell returned to deal with several blocks, and to regularise any unevenness in the completed surface. Helmet-diver working was limited to tidal velocities below 1 knot, restricting operations to about 4-5 hours each tide, during which 6 blocks were placed per hour at best.

Trimming and filling the 'Low-water course' compensated for any errors in lower layers. Blocks above were set by masons during the 2-3 hours of low water on spring tides. All the upper courses were set and bedded in 2:1 Portland cement mortar. All lower joints were caulked by sacking and/or rope, pointed in neat (quick-setting) cement, to avoid any loss of the jointing or bedding mortar downward.

Toe protection blocks were laid along the seaward face using essentially similar procedures with a smaller diving bell operated from a luffing-jib crane running along the wall. As these protection aprons were completed, so the parapet walls were added above. A capping layer of in-situ concrete with granite paving completed the deck, allowing for rails, gas / electric / telephone cables and water pipes.

The extended harbour was opened by the Prince of Wales in November 1909. Wilson (1919) gives the cost of the expansion as £3,500,000.

On declaration of war in 1914, ferry and commercial activities moved to Folkestone, Dover reverting to naval use. After the war, the harbour remained with the Admiralty but the Commercial Harbour was managed by Dover Harbour Board (DHB) who had to deal with years of neglect and adaptations. Ferry and commercial trade increased, and in September 1923, Admiralty Harbour was transferred by Act of Parliament to DHB, with the Admiralty reserving that the harbour would revert to the Admiralty should Defence of the Realm require.

In 1931, Southern Railway launched a car-ferry, their first designated cross-Channel car-carrying ferry. From the mid-1930s cross-Channel passengers and cars increased rapidly, as did freight. Plans were made to increase the number of berths to use the Eastern Dockyard. In September 1939 Admiralty Pier with the rest of the harbour, came under the Admiralty as part of Fortress Dover. After the war, in November 1949, DHB promoted a Parliamentary Bill to create a car ferry terminal at the Eastern Dockyard for the bulk of passenger services. Previously railway ferries from Admiralty Pier had dominated ferry traffic. Most such traffic has since moved to the Eastern Docks. Dover Harbour now remains the main route for UK roll-on roll-off trade, lorries, coaches and private vehicles.

## 2.14 Closing remarks

This chapter has highlighted key example breakwaters constructed over the period of interest, touching on some of the factors that led to their failures or survival.

Theories used in design of breakwaters constructed in this era suggested that the rubble foundation should not be brought above the depth where it could be disturbed by wave action. This depth was often judged or refined over the first few winters from the start of mound construction. In severe wave conditions, material might be lost from the mound quite rapidly if it had been placed too high. As experience grew, predictions of a safe level for the mound became a little more certain, although judgements varied, as will be shown in the discussions in Chapter 3. This simplistic approach however suffered two major flaws.

Firstly, it ignored the influence that the wave reflected back from the wall itself has on the mound material. At some sites, this effect may not itself have been very severe, and settlement or consolidation of the mound might have generated a suitable increase in stability of the mound. At other sites however (e.g. Alderney or Aberdeen) the wall was high, wave attack severe, and erosion or any movement of the mound could lead quickly to loss of support to the blockwork.

Secondly, this empirical approach relied on storm conditions during the early part of construction being representative of conditions during the rest of the structure life. The rate of construction progress was relatively slow (construction periods of 5+ years were quite common) so this limitation was less important than it would be now.

Once sufficient seaward extent of mound had consolidated, the blockwork walls were then constructed to resist and reflect the incident waves. Vertical or steeply battered sections allowed most of the weight of overlying blocks to increase frictional restraint to the lower blocks which would otherwise be "sucked out" from the wall, see the discussion in section 3.5 and the model tests described in Chapter 7.

The multiple failures of the breakwater wall at Alderney (sections 2.7 and 6.3), particularly in the early stages of construction, and that at Greve de Lecq (section 2.12.2) confirm how essential it was to secure toe blocks against even small movements that would allow the masonry above to lose bond by unlocking the downward ‘pinching’ forces.

To estimate the stability of selected breakwaters, a series of empirical analysis tools developed mainly over the author’s career have been applied in Chapter 6 to three case studies at Wick, Alderney and Dover. The wave load and Safety Factor analysis will describe representative wave loads, principally momentum-driven loads, and then estimate sliding and overturning resistance to give the Factors of Safety. The analysis of overturning also acts as a proxy for loosening of the blockwork face, potentially leading to loss of bonding and then dis-aggregation of the face.

# 3 STRUCTURE CONFIGURATIONS, AND MATERIALS

## 3.1 Introduction

Having introduced example breakwaters in Chapter 2, Chapter 3 describes the main breakwater configurations use in the study period, their component parts and materials, and discusses the influence of these on overall and/or local stability.

In theory, timber discussed in section 3.2 is easy to use, may have been plentiful in the period of interest, and can carry tensile as well as compressive forces.

The most common breakwater types described earlier were formed by a submerged rubble mound acting as a foundation for a masonry wall, section 3.3. A short diversion in section 3.4 describes quarrying operations that produced both rubble, and stone blocks. The stability or otherwise of blockwork walls under wave attack is then discussed in section 3.5.

Having highlighted many of the sources of damage or failure of such walls, sections 3.6 and 3.7 describe replacement of stone blocks by concrete blocks, and show how those components allowed wall foundation levels to be lowered. The alternative approach of forming the wall foundation using concrete in fabric bags is described in section 3.8.

## 3.2 Timber

Whilst a few early breakwaters used timber to retain dumped stone, see section 2.1, attack by boring worms, rot and decay has left none intact unless later encased. The early lengths of Dublin Great South Wall had to be replaced or encased in stone from about 1753, see section 2.2. In his book discussed in section 11.8, Stevenson (1874) discusses damage by worm attack and rot at some length with a notable table of durabilities of some 30-40 types of timber and/or treatments at Bell Rock. He identifies only Greenheart, Teakwood or Beefwood as having any significant durability (more than 10 years). Most other timbers had damaged badly or failed within 5 years of exposure. As well as testing un-treated timber, Stevenson discusses timber treatments such as creosote (then recently patented).

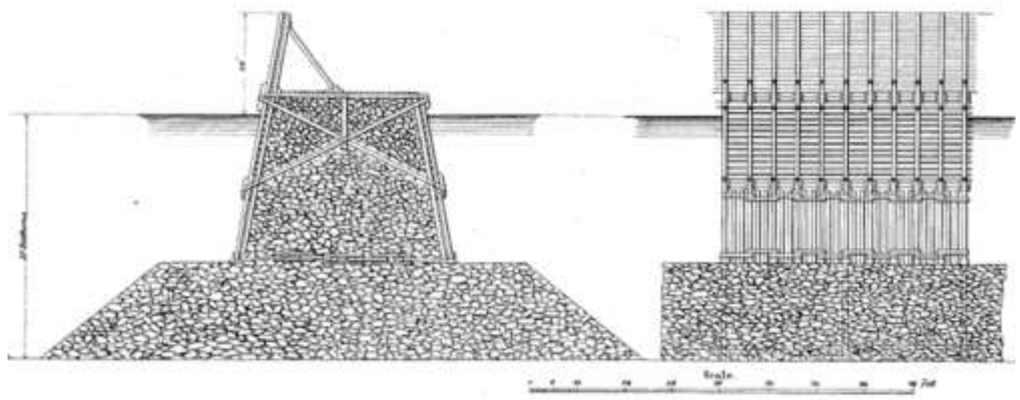

**Figure 3.1 Timber crib breakwater**
*After Scott (1858)*

Despite the likelihood of decay, Stevenson does emphasise the advantages imparted where timber can be used to provide tensile reinforcement to masonry.

The example of use of timber in Figure 3.1 suggested by Scott (1858) was roundly criticised in the discussions following his ICE presentation (see Chapter 11) and was not adopted on site. The timber cones at Cherbourg discussed previously in section 2.9 and shown here in Figure 3.2 were however installed, but were unsuccessful, and the remains were later buried within the rubble mound.

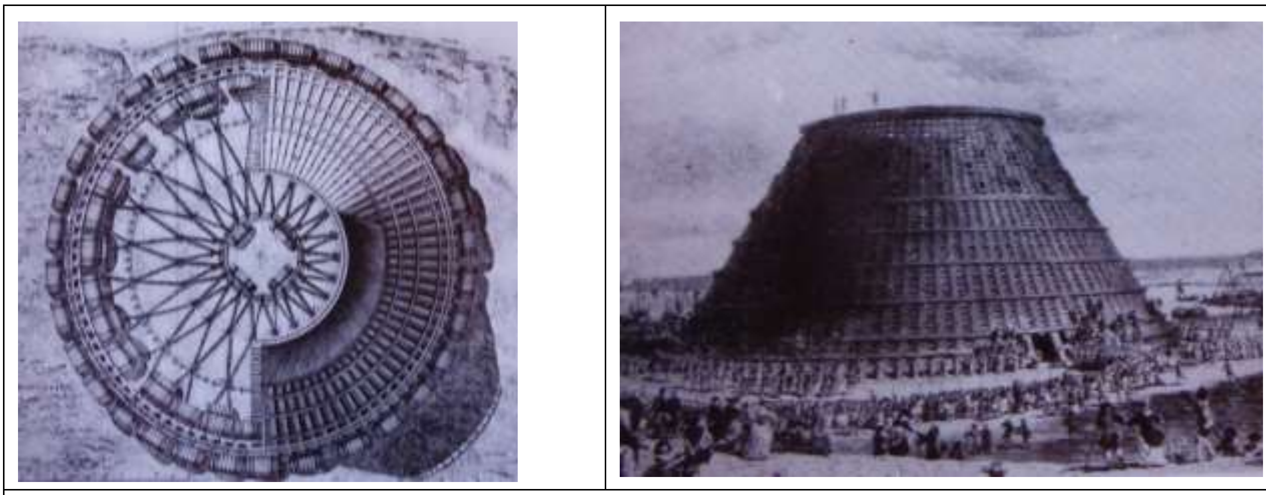

**Figure 3.2 Timber cones used at Cherbourg**
*From Cachin (1820)*

A further type of inclined wave screen discussed in Chapter 11 that was not adopted is shown in Figure 3.3, where the screen elements were proposed to be formed of iron rails. It is not believed that this was proceeded with, and it was certainly roundly criticised by John Scott Russell in the discussion on Scott's second paper on Blyth (1860), see also section 11.7.[18]

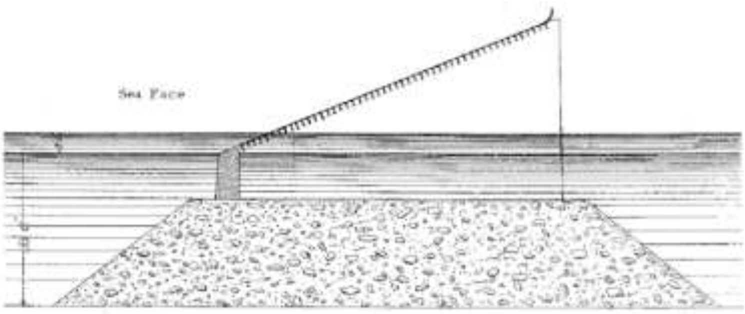


**Figure 3.3 Proposed Table Bay breakwater wave screen**
*From Scott (1860)*

[18] In recent years, timber wave screens supported on piles as in Figure 3.4 have been used to protect marinas against direct or reflected wave attack. The hydraulic performance of such wave screens was discussed by Allsop & Hettiarachchi (1989), and the practical design and construction by Gardner *et al* (1986, 1988). Planks of tropical hardwoods may form the wave screen panels. Even so, rot and decay of the timber has often forced replacement of those elements by concrete panels or by planks of reinforced concrete or recycled plastic.

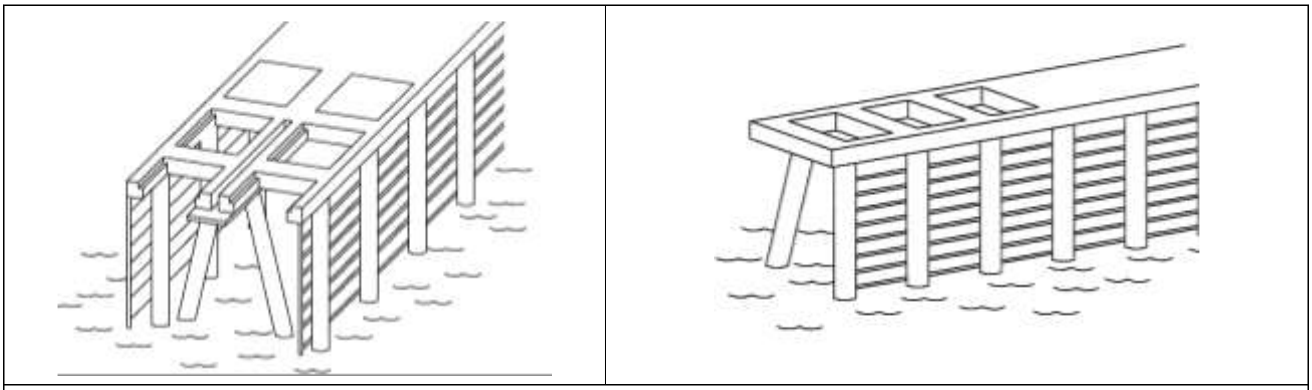

**Figure 3.4 Double and single screens (timber panels**

## 3.3 Masonry walls on rubble mounds

The great majority of breakwaters of interest in this research have used masonry walls on a rubble mound foundation, as shown in the example in Figure 3.5.

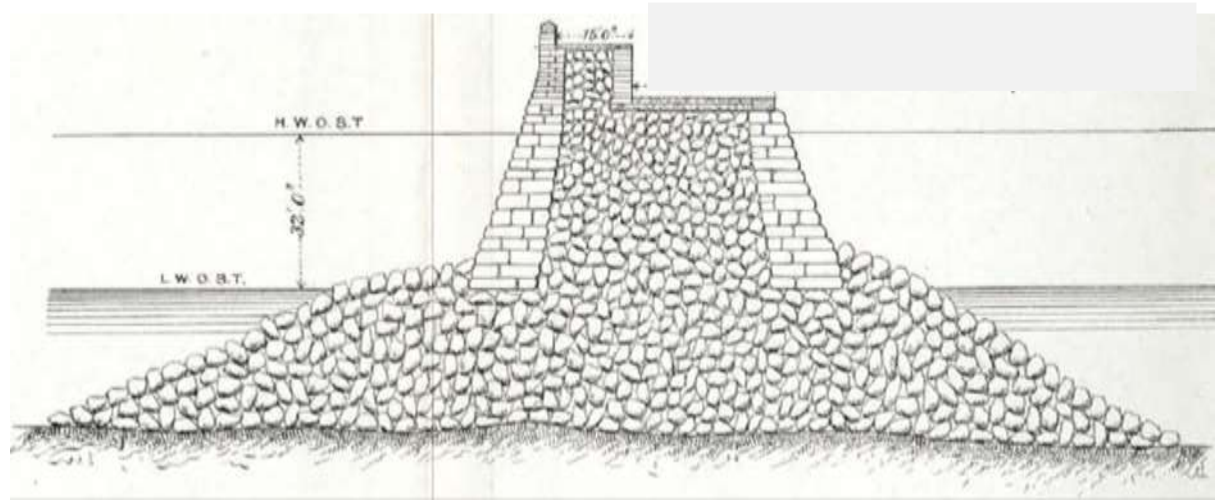


**Figure 3.5 St Catherine's overall section, 1847**
*After Vernon-Harcourt (1885)*

The key features for such structures designed before about 1850 may be summarised:

a) The mound was taken up to a level close to LW so that masons could place the foundation stones without assistance of divers.
b) The mound was formed to side slopes and to a crest level that (it was hoped) were stable under frequent wave action. Indeed, the mound was often left for some time to be 'in equilibrium' with incident wave action.
c) The superstructure is formed by masonry walls, filled by rubble, often including fines generated from quarry / site operations to shape the stone blocks.
d) Cementing of blocks was limited, usually only those above-water levels for which the (lime) mortars could resist wash-out.
e) The upper superstructure on the seaward side reduced wave overtopping, but also added weight to assist 'pinch' wall blocks together.
f) The crest was paved for traffic, and to resist overtopping forces.

In discussing such structures, Allsop (2009) summarised the main failure modes which such structures were required to resist:

a) Sliding or overturning of the breakwater wall as a single entity;

b) Geotechnical failure of the mound, allowing movement of the wall;

c) Removal of blocks from the wall, resulting in loss of continuity;

d) Local failure of the mound releasing restraint to blocks, leading to loss of fill and/or loss of continuity of the blockwork.

It is important to note that these failure modes are not necessarily alternatives, but may contribute together in varying proportions to failures. The main failure modes of sliding or overturning are discussed briefly in section 3.5 later in this chapter. The main stability analysis methods are described in section 5.7, and are then applied in the case studies in Chapter 6.

## 3.4 Quarrying rock

Construction of large rubble mounds, and of blockwork walls, all require substantial volumes of quarry rock in useful sizes, and (some of it) capable of being shaped to form suitable blocks for masonry.

A little curiously, the quarrying and preparation of stone blocks is seldom discussed in the literature, indeed the handbook by Vernon-Harcourt appears to be entirely silent on the issue. In contrast, Shield (1895) reviewed in section 11.11 devotes all of his Chapter VI to quarrying, although he aims merely "*to direct attention to such points as ... may be of use to the young harbour engineer.*"

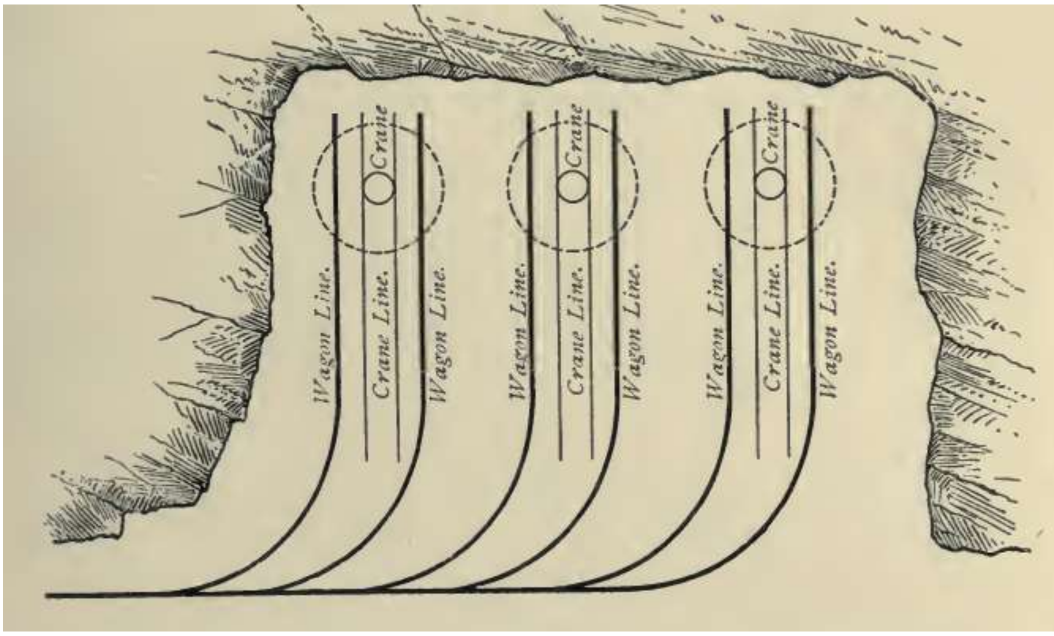


**Figure 3.6 Quarry layout suggestion by Shield (1895)**

In selecting a quarry site, Shield advocates use of "*a sufficient number of trial pits ... to furnish an accurate knowledge of the ground.*" He notes that overburden must be disposed of, requiring additional space and increasing costs. Shield reminds the reader that such disposal requires drainage, naturally or by pumping.

On quarry layout, he anticipates use of railways to move material out, and cranes to extract and load rock, see example layout in Figure 3.6. To handle rock from the blast, Shield recommends use of powerful cranes, capable of moving 10-15t rocks at minimum.

For quarry blasting, Shield gives suggested coefficients to determine the quantities of blasting powder relative to the rock type and "*the line of least resistance*", to be assessed taking account of any faults in the rock. Shield does separately acknowledge use of nitro-glycerine, invented by the Italian chemist Ascanio Sobrero in 1846, and dynamite, patented by Alfred Nobel in 1867.

Shield gives examples of blast yield for chalk quarrying at Dover, yielding about 85 tons of rock per kg of powder, but in hard rock decreasing to about 11 tons of stone per kg of explosive. Shield generally advocates vertical holes with the powder carefully tamped to ensure that the blast is outwards, not upwards. To bore the holes, Shield describes manual boring using short "*jumpers*" or 2m long drills, generally handles / struck by teams of 3 men. He notes that drilling at Holyhead was by hand, but at Port Elizabeth (South Africa) steam-powered Ingersoll drills were used. He compares rates of drilling (by hand) at Holyhead, Port Elizabeth, Peterhead, Dalkey near Dublin, and near Plymouth, giving depths per hour for teams of 3 men in hard rock varying from 2m per hour in limestone, 0.3-0.6m per hour in quartzite or granite, to 0.1-0.2m per hour in very hard rock.

In discussing explosives, Shield particularly notes the effect of blast fractionation where the effects of "*quick explosives*" may propagate deep into the working face, which might be acceptable where rubble is to be produced, but not where dressed stone (ashlar) is to be produced.

Stevenson (1874) does not discuss quarrying to the extent that Shield does, but Stevenson does however give advice on dressing of stone blocks, which would often be part of the quarrying activity, and on their later fitting in masonry walls. Stevenson particularly summarises his advice to <u>not</u> import into marine design / construction "*the laws and maxims of house-architecture, with its careful vertical bond, and its small but finely-dressed face-stones*". He has more advice on 'dressing' of stone, and the dangers in reducing friction between blocks if they are finely dressed or polished cautioning against "*the evils of fine workmanship*". He tabulates coefficients of friction ($\mu$) in air and water for various surface treatments, also quoting experiments by George Rennie who measured a coefficient of friction, $\mu = 0.8$ for rough stones pulled over a quarry floor.

Stevenson was not concerned over protuberances from the face-work, and he advises to keep the beds (horizontal courses) rough. He also advises that backing material should be "*carefully set and regularly bonded with the face-work*".

For roadways atop the breakwater wall, Stevenson acknowledges a diversity of opinion as to whether the roadway surface should be of dry masonry (not cemented) allowing it to vent air pressures from within the fill below, or to be "*rendered impervious*" with air discharged perhaps through the "*mooring pauls*" (hollow-centre cast iron bollards).

## 3.5 Stability / movement of wall elements

The stability of blockwork walls under wave attack depends on many factors, often site-specific, and indeed often localised along a particular length of structure. Local wave pressures (see Chapter 5, particularly section 5.5 on impulsive loads) will therefore be extremely variable. Individual blocks are themselves small relative to wave heights and wavelengths, and are therefore relatively light relative to disturbing wave forces. The net consequence is that there are no established methods to predict blockwork stability, other than custom and practice.

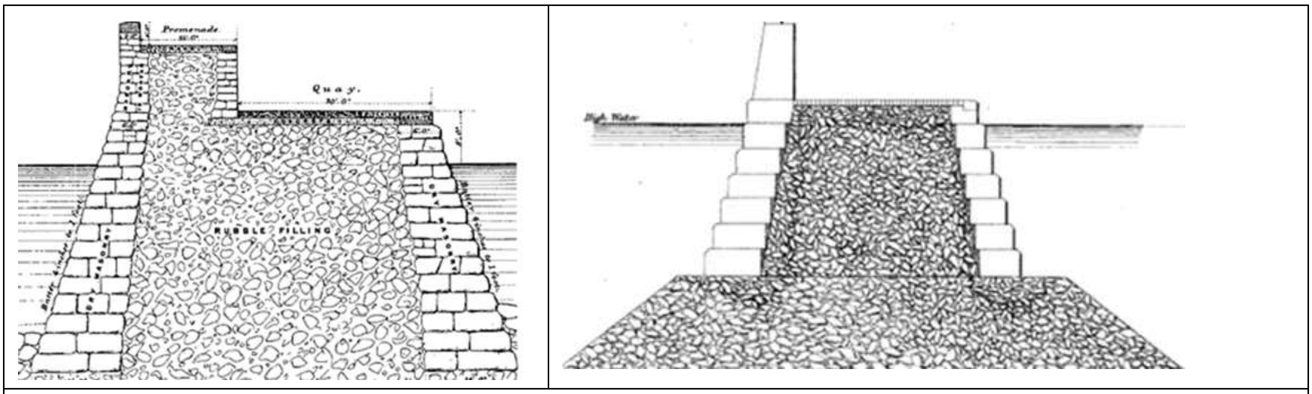

**Figure 3.7 Blockwork breakwaters at: a) St Catherine's, Jersey; b) Blyth**
*After Vernon-Harcourt (1885); Scott (1860)*

For breakwaters with monolithic walls, the main failures may be by sliding (seaward or landward), or by overturning, For the blockwork breakwaters considered in this thesis, sliding or overturning of monolithic elements or gross foundation failure have been relatively rare in the UK, although not unknown *viz* the monolithic outer end at Wick, see section 6.2.

Local failures leading to loss of continuity, and thence to overall failure, have been much more common, although these local failures might be stimulated as (or when) the overall structure approaches conditions under which a monolithic structure would be at risk of sliding or overturning failure. Incipient overturning failure may be particularly linked to the extraction of individual blocks. As overturning moments increases, and a blockwork wall approaches overturning, the net weight of the outer part of the wall reduces. Within the wall, confining or 'pinching' forces in the front face will reduce, thus reducing the forces retaining individual blocks. If those blocks are subject to impulsive loadings, those less-restrained blocks may move. Examples of the failures of breakwaters at Alderney or Greve de Lecq have been discussed in Chapter 2, and overall safety factors for Alderney and Wick are analysed later in Chapter 6.

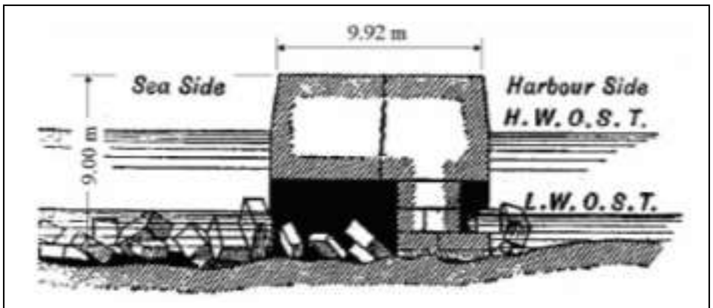


**Figure 3.8 Blockwork damage at Aberdeen**
*From Marth et al (2005) after Shield (1895)*

In general, direct wave impact pressures will not cause distress to good quality stone blocks *per se*, although repeated impacts may lead to deterioration of the stone. Small movements in dry-jointed blockwork will allow small particles to enter loose joints, and may jam them open, or may lock them tight. Mortar joints will however gradually deteriorate as they lose strength and fine materials are washed out. This will accelerate where wave impacts are sufficiently heavy to cause small and/or local movements of the wall, or where the foundation mound moves, see the discussion on Greve de Lecq in section 2.13.

In time, wave pressures penetrate around any loose block, and will reach the hearting material behind the face. Experience has shown that voids then develop within any wall subject to frequent impacts, often being unseen by any inspecting engineer until the upper roadway collapses. The example of the local voiding at the head of St Catherine's breakwater was discussed by Hold (2009).

A further example is provided by Marth *et al* (2005) who discuss a failure at Aberdeen recorded by Shield (1895). They note that dry stacked (concrete) blocks of ~11t were surmounted by a 4.8m thick in-situ concrete superstructure, Figure 3.8. Marth *et al* (2005) note that seabed settlement could not be followed by the stiff superstructure, opening up the space between foundation and superstructure, and thus reducing the pinching forces securing the lower blocks in the seaward face. Marth *et al* note that blocks were also displaced from the leeward face as shown in Figure 3.8, probably caused by transmission of impulsive pressures through gaps between the remaining blocks.

Over the period of interest to this research, no methods were available to predict wave forces on blockwork, or the restraining forces. Recently however, Marth *et al* (2005) discuss the effects of 'pinching forces' on blocks in the Alderney breakwater wall using photo-elastic materials to illustrate example stress paths, Figure 3.9. In doing so we noted local instances of absence of load from above, indicating that pinching forces will have been reduced, perhaps to zero. At this point the sole mechanism to reduce outward wave pressures will be the block's own weight, and any friction with adjoining blocks.

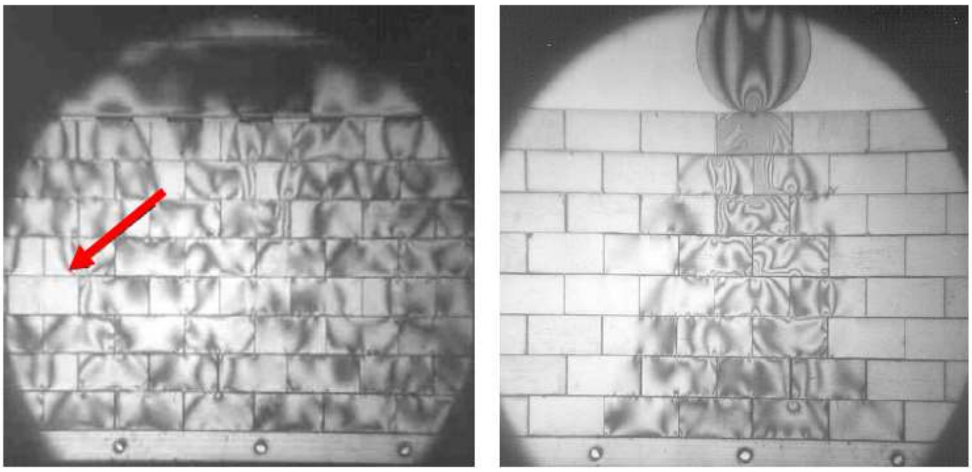

**Figure 3.9 Stress paths in photo-elastic material**
*From Marth et al (2004) after Klavzar (2004)*
*Red arrow indicates a block with no load passing through it, i.e. lacking any 'pinching' force.*

This problem of block extraction was first analysed by Allsop & Bray (1994) who considered a single block in a wall, 2m deep x 1m across x 1m high, in the main wave impact zone slightly above water level attacked by breaking waves of $H_b$=5m. The analysis postulated a void behind the block, it being restrained only by its (buoyant) self-weight, and frictional resistance against the block below. In that paper, the magnitudes of impact pressures acting on the front face were estimated using formulae by Partenscky (1988), discussed further in Chapter 5.

**Figure 3.10 Extraction of a single block in a blockwork wall**
*From Marth et al (2004) Model tests at 1:20 scale.*

Estimating that the impact pressures on the front face is now transmitted to the void behind the block, in un-aerated water, the impact pulse will reach the back of the block in ~0.002s. That is not likely as any wave hitting the wall would include air entrained from previous impacts. Taking air contents between 10 and 30%, the speed of sound in the air / water mixture drops to approximately 30m/s, slowing transmission of the pressure pulse to 0.07s. At the time that this pulse causes the pressure behind the block to reach its peak, the pressure acting on the front face will have fallen significantly, even turning negative in some instances. The differential pressure outward may then reach a significant proportion of the original impact pressure. Even if severely damped, a net outward pressure of 0.5 to 0.2 times the original impact pressure would not be unreasonable.[19]

In this simplified analysis, the block is restrained by its self-weight, in this example of 5.3 tonnes, 3.2 tonnes in water, giving a frictional resistance of 2.2 tonne [$\mu \approx 0.7$.]. The force needed to overcome this would be generated by a net pressure difference across the block of ~22 kPa. Allsop & Bray (2004) estimated that for well-aerated waves, the pressure on the front face could well exceed 600 kPa, perhaps up to 3000 kPa under more pessimistic assumptions. Even considering the lower pressure, the net outward pressure might easily exceed 100 kPa, suggesting that even mild wave impacts might move a loose block.

A rather similar analysis can be derived from the 1:20 scale model test example discussed by Marth *et al* (2004) who considered a block of end area (model units) 20mm x 80mm, and length into the wall of 100mm. Such a model concrete block might have a mass of 0.38kg, giving a frictional resistance for a loose block of 2.6N, equivalent to a pressure on the rear face of the block of 1.6kPa. Using their estimate of internal pressures up to 0.25 x external pressures, this is equivalent to an external pressure of (say) 6.5kPa, equivalent to 130kPa at full scale.

In their model tests loosely based on a cross-section of Alderney breakwater, Marth *et al* (2004) developed example pressures of 25kPa (equivalent to 500kPa at full scale) and quote an upper limit of 70kPa (1400kPa at full scale). They then show photographs of a single block in the process of being extracted from such a wall, claimed as by a single wave impact, see Figure 3.10.

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[19] The initial analysis by Allsop & Bray (1994) was confirmed 10 years later by Marth *et al,* (2004) who estimated that internal pressures acting outward might reach 0.25 of the external pressures.

## 3.6 Concrete and concrete blocks

Cutting stone blocks to size and shape to form blockwork walls was slow and laborious, and required significant skill. The advantages in structural connectivity were however seen where stone blocks were shaped (expensively) for lighthouses, see Stevenson (1848, 1874), Bathurst (2000) and Nancollas (2019). A slightly simpler approach was to use iron bars chased into the stone and secured by pouring in molten lead, or by cement grout. Cutting the channels increased the complexity and costs of stone preparation, as well cutting and shaping the iron bars to fit, and of melting and pouring the molten lead.

It was therefore logical that designers would seek to replace 'hewn' blocks by pre-cast and shaped concrete blocks when that became possible. The complications and cost of conventional stone blocks could be substantially alleviated by manufacturing blocks to given (generally uniform) sizes, and with 'male' or 'female' key-ways already cast in.

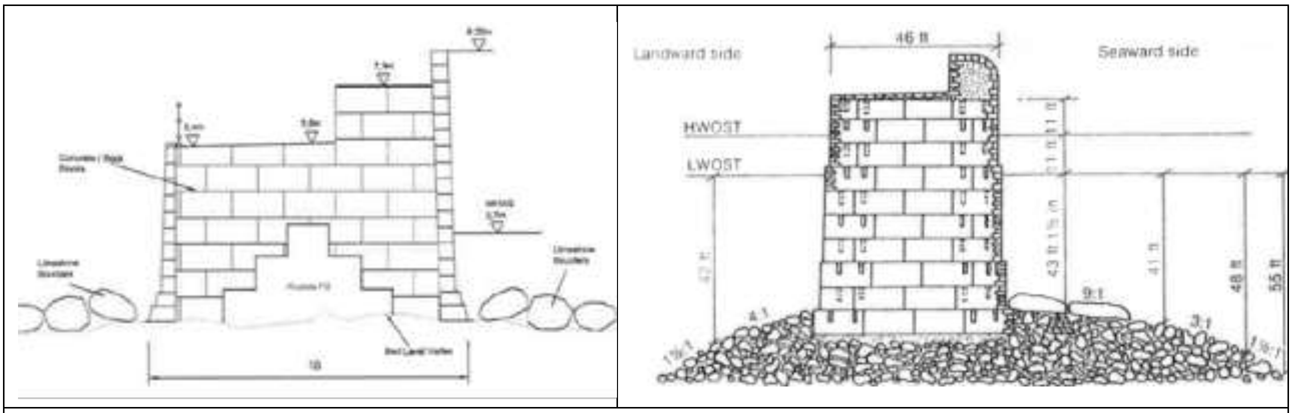



**Figure 3.11 Blockwork breakwaters at: (left) Hartlepool, (right) Peterhead**
*After Hampshire et al (2013); Buchan (1984)*

Various cements had been used historically by the Greeks and Romans in the Mediterranean, see Franco & Verdesi (1993), but 'hydraulic' cements that could set underwater and resist exposure to wave action and abrasion were rare, requiring use of naturally 'pozzolanic' materials. The development and testing of early cements are discussed later in section 14.4.

Whilst a number of patented cements appeared beforehand, Portland Cement (later Ordinary Portland Cement, OPC) was developed and patented by Joseph Aspdin in 1824. That cement was however only available commercially in bulk production by 1840-50. As an example, Wouldham Court Cement Works in Kent used by Pearson for the Dover outer breakwater contract had only opened in 1847.

Whilst the single skin wall construction at St Catherine's or Blyth (Figure 3.7) might survive in mild wave climates, perhaps strongly depth-limited conditions, the availability of concrete to form uniform blocks at some time after 1860 allowed designers to replace erodible fill by bonded concrete blocks. This replacement of rubble fill was adopted for the 'new' breakwaters at the river entrances to the Tyne and Tees, and for the Heugh at Hartlepool, and for the Harbours of Refuge at Peterhead, Figure 3.11, and Dover, Figure 2.34.

The construction at Dover has already been discussed in section 2.13, which high-lighted the use of joggle joints to transfer loads across joints. Section 2.13 also noted the use of iron rails concreted into chased channels to give tensile strength at the outer ends of the breakwater.

## 3.7 Slice blockwork

Historically, on larger breakwaters masonry was most commonly laid in horizontal courses. But in many smaller breakwaters, particularly where the local rock was thinner-bedded, masons laid blocks on edge (examples in Figure 3.12). Apart from the strong recommendation by Stevenson (1874), see below, in favour of slice blockwork, other literature does not seem to suggest any particular reasons for preferring horizontal or vertical courses, perhaps local practice. Slice blockwork is often ascribed to Telford or to Smeaton (see Chapter 12).

**Figure 3.12 Variable blockwork placement at: a) Cockenzie; b) St Andrews**

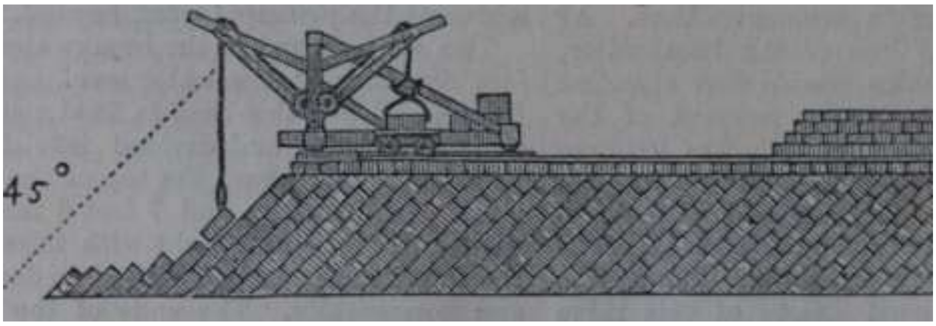



**Figure 3.13 Slice blockwork**

*After Bartholomew (1870) citing Telford's 1810 extension to Aberdeen*

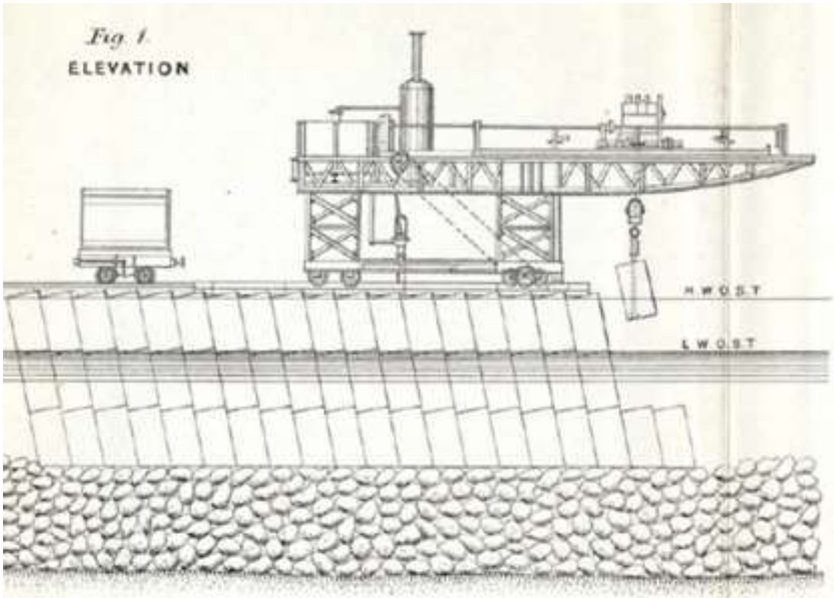



**Figure 3.14 Slice blockwork**

*After Vernon-Harcourt (1885)*

Stevenson (1874) discusses "*the method of assembling stones on edge, instead of on their beds, which was used in some old Scottish harbours and seawalls, as at St Andrews, Prestonpans, etc., deserves to be more generally known and adopted from its greatly superior strength*" a pretty strong endorsement! Stevenson

quotes James Bremner of Wick as supporting this approach to help force blocks together. Stevenson concludes "*I do not hesitate to assert that it is a great engineering error to assemble stones in exposed works in any other way than on their edges, and I extend this remark even to materials of ordinary thickness, although the advantage is most conspicuous where the materials are thin*." He does however caution that this approach must not be used "*where there is any risk of heavy seas coming in the wrong direction so as to strike the masonry on the overhanging side*." In Bartholomew's simple sketch in Figure 3.13, slice work might safely be used for wave attack from the left, but not from the right.

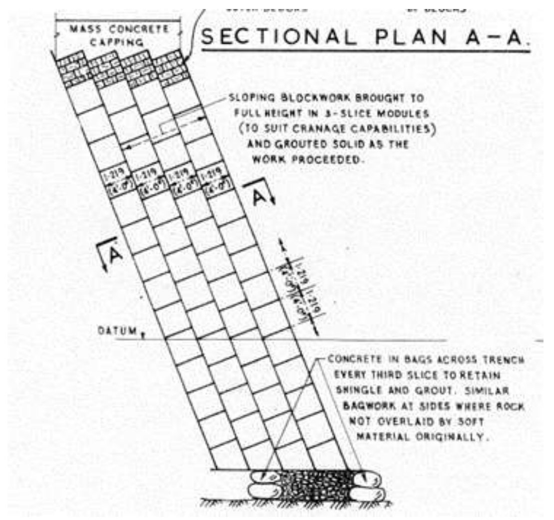



**Figure 3.15 Slice blockwork extending the Hermitage Breakwater in 1887-88**
*Courtesy Jersey Harbour Master*

Stevenson (1874) then speculates that the masonry strength might be further increased if the stones were to be dressed to rhomboidal form, allowing the top and bottom faces to be horizontal whilst bedding sides were on the slant.

The vertical (or steeply inclined) block placement was probably adopted when the foundation was variable in level and/or potential settlement. Bartholomew (1870) reviewed in Chapter 14 cites Telford as the inspiration for inclined slice-work.

Rather later arrangements using concrete blocks are shown by Vernon-Harcourt (1885) in Figure 3.14 and from a Coode design for the extension of Hermitage breakwater at St Helier, Jersey in 1887-88 shown in Figure 3.15.

This approach is however relatively rare, being generally confined to older and remoter breakwaters of Scotland. The approach met with some support when blocks could be pre-cast in concrete (see above and comments by Mike Leonard of Coode & Partners in section 13.15), but the technique was still relatively rarely used.

## 3.8 Mass concrete foundations using bags

Forming the lower parts of a breakwater wall in mass concrete was fraught with the difficulties in securing the form-work, especially under wave attack, and protecting the fresh concrete against wash-out by waves or currents. A novel technique of placing 'liquid' concrete in large bags was used by W Dyce Cay at Aberdeen, where the new South Breakwater was completed in autumn of 1873. (Cay's paper 1389 was read to ICE in December 1874). This breakwater appears to have been the first use of large concrete-filled bags to form the foundation layers. The foundation was then followed by concrete blocks, 9-24t each, then topped by a mass-concrete upper section.

The foundation was formed by bags filled (initially) by 5.25t of concrete, and (later) of 16t. These were placed from bottom-opening iron skips, lowered under the supervision of divers, the bag being discharged when the skip was over the required position. Construction staging used 68 Oregon pine masts or piles, each 20m long in the finished work. Travelling carriages carried 25t steam Goliaths by Stothert & Pitt, further discussion on these and other cranes is given in section 14.2. A 3t steam derrick crane was used to erect staging. Cay comments on the steam Goliaths that "*the economy in working it more than paid for the additional first cost of the steam power*".

In the first year of construction, mass-concrete bay lengths varied around 5-6 m, extended in the second year to 5-9m long, the increase being achieved by including large concrete 'plums' (up to 25%) in the in-situ concrete. The formwork (generally?) used sacrificial iron tie rods across the construction. Concrete mixing used four Messent machines, each mixing 0.4m$^3$ at a time, and delivering 9m$^3$ per hour.

Around the toe of the wall at Aberdeen, large (100t) bags of concrete for the toe protection were dropped from a pine hopper box suspended from brackets mounted on the upper wall.

In discussing his own paper, Cay (1874) highlighted his intentions for the planned extension of Aberdeen North Pier. The use of large concrete-filled bags would be continued, now larger at 50t each. Cay then discusses relative costs of hopper barges for 50t and 100t bags.

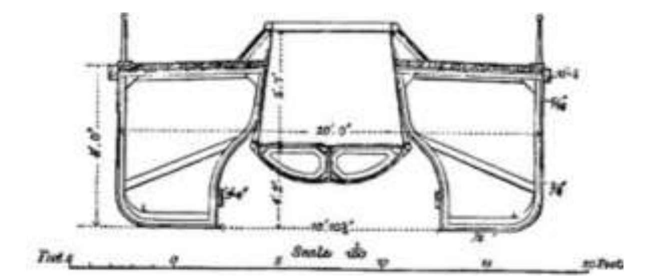


**Figure 3.16 Section through a bottom-dumping barge for concrete–filled jute bags**
*From Vernon-Harcourt (1885) and Carey (1886)*

On the proposed works at Newhaven, Sussex, Carey (1886) intended to found the essentially monolithic breakwater on large (100t) concrete bags based on the system developed by William Dyce Cay, see: Cay (1874), Vernon-Harcourt (1885), and Turner (1986). In this instance the jute / canvas bags were rendered water-tight by coating inside with marine glue.

A new steam-driven hopper-barge was commissioned from Simmons & Co of Renfrew to hold and deposit the 100t bags. The hopper was reverse tapered (wider at bottom than top), and the doors were designed not to snag the falling bags (Figure 3.16 and Figure 3.17). A new concrete batching and mixing machine was devised by Carey and Latham, constructed to ensure that 100t of concrete was fresh, well mixed, and delivered into the bag within 20 minutes. Gravel and sand for the concrete were gained from the beaches. For the main works, a further continuous mixing machine was devised to output 55m$^3$/hour.

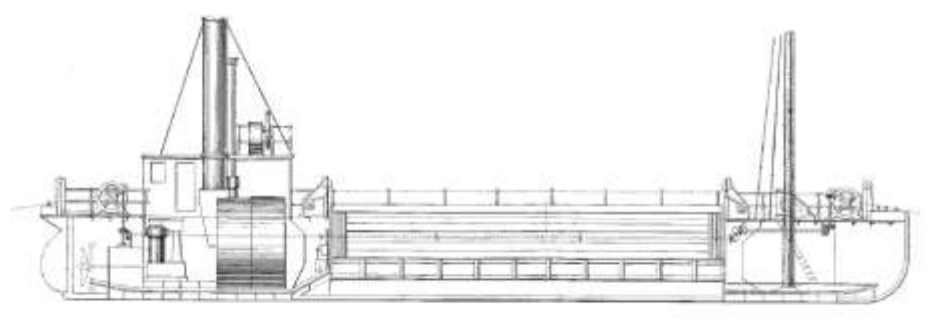


**Figure 3.17 Bottom-dumping barge for concrete – filled bags**
*After Carey (1886)*

In the discussion to Carey (1886), Messent cautioned that forming the main structure in concrete-filled bags was still strongly weather sensitive, whereas building concrete blocks could take place even when on-site construction was not possible.

In a later discussion by Carey to a paper on construction plant by Pitt (1893) reviewed in 14.2, use of bags up to 160t is illustrated in Figure 3.18.

It is probable that this use of fabric bags allowed construction to advance relatively rapidly, but concrete might have been rather variable, and its durability therefore similarly variable. Filling and placement of the large bags was weather dependent. The need for this method of dealing with uneven foundations was probably overcome by use of large concrete blocks placed by Titan or Goliath cranes, see section 14.2, assisted by increased used of helmet divers, as discussed in section 4.2.

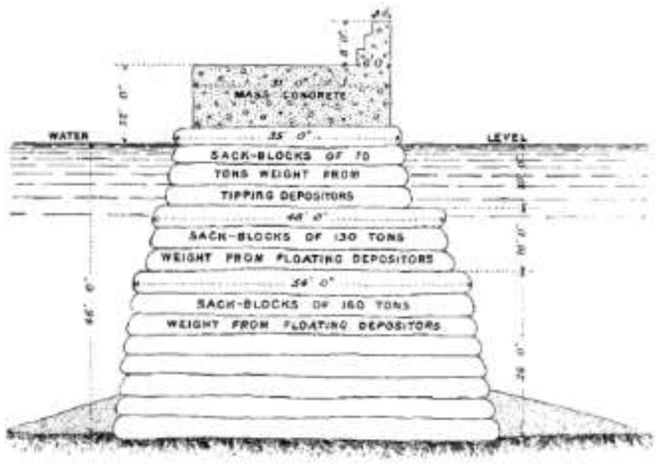



**Figure 3.18 La Guaria Breakwater, Venezuela**
*From discussion by Carey to Pitt (1893)*

# 4 CONSTRUCTION

## 4.1 Construction technologies

The simplest methods of construction for simple vertical breakwaters involve land-based plant, initially manual or horse-drawn, see example in Figure 4.1. Initially, and particularly for small and simple breakwaters in shallow water, then the delivery could be along the partially constructed mound or wall. It was however rapidly appreciated that the mound needed to be kept low enough to avoid causing wave breaking onto the super-structure, thus obviating using the mound crest as a working surface.

The simplest approach was then to deliver stone for the mound by boat, in any case necessary for any shore-detached breakwaters. For a breakwater of any size, this would require many vessels, around 60 were used at Plymouth, see section 2.12. Examples of stone-dumping from rowed or sailed boats for Cherbourg were shown earlier in Figure 2.25. In the early 1800s, steam power was not available, either for vessels, or to pull wagons, or for lifting.

**Figure 4.1 Use of horse-drawn wagons on staging at Alderney**

The benefits of purpose-built staging, rather than using the breakwater wall itself to provide the base for construction, were hotly debated in ICE discussions through this period, see Chapter 11. Both approaches had their advocates, although the use of staging (albeit temporary) appeared to grow in popularity as its advantages became clearer in reducing to some degree the weather-dependency of construction operations. A relatively late example of running block-placing cranes along the 'finished' work is illustrated for Peterhead in Figure 4.2, probably dating from 1892, see discussion by Buchan (1984).

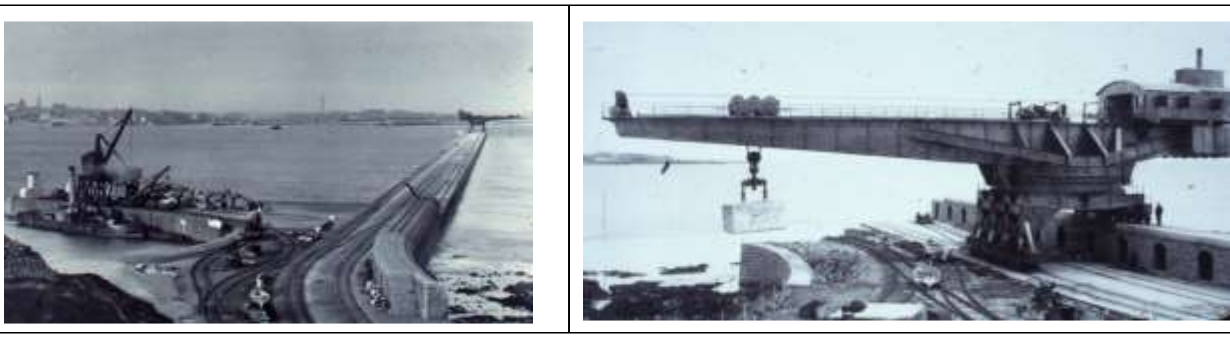


**Figure 4.2 Peterhead South Breakwater, construction around 1892**
*Courtesy of Peterhead Bay Authority*

The alternative approach using staging depended on the availability of timber in sufficient lengths, and durability, often shipped from tropical countries. At Aberdeen, Cay (1874) used 68 Oregon pine masts (piles), each 20m long in the finished work. At Dover, the (mostly Tasmanian Blue Gum) piles were upwards of 30m long, of ~ 0.5m square section. Wilson (1919) notes that Blue Gum timber was heavier than water, but that Oregon pine when used required weighting by old iron rails to ensure sinking. Staging piles were re-used as the work progressed, being extracted by a floating hulk with a jib and winch. After use, piles were spliced to ensure availability of an undamaged head for driving. Where the ground was not conducive to driving, piles could be tipped by Mitchell's patent screw, see Figure 14.4.

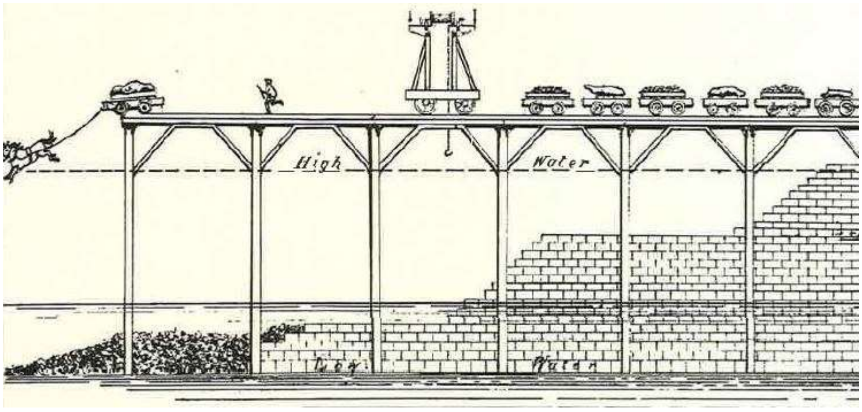



**Figure 4.3 St Catherine's - Delivery of stone to foundation[20]**
*After Davies (1983)*

Earlier examples of timber staging are shown in Figure 4.3 and Figure 4.4 respectively for St Catherine's and Alderney, both constructed from 1847 onward. It is interesting to note the cross-bracing to reduce sway, attached to the piles above HW.

In the lower tidal range, but much greater wave attack at Alderney, an innovative adaptation was used to take the rail-delivered rock from the staging to the dumping barges without shooting the rock straight through the bottom of the barge! The reverse flow arrangement in Figure 4.4 slowed the fall velocity enough to avoid sinking the receiving barge.

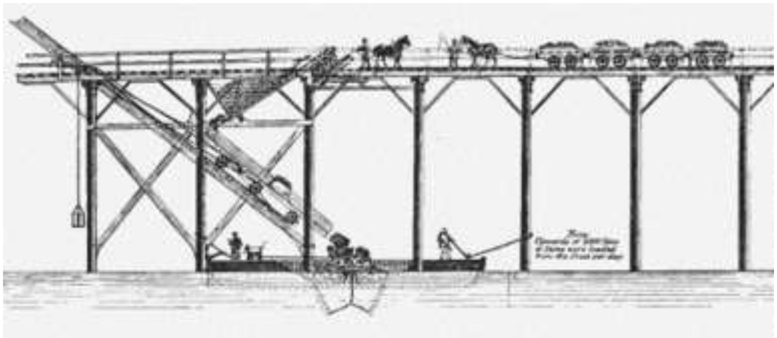


**Figure 4.4 Alderney - Delivery of stone to barges by Jackson & Bean's shoot**
*From the Industrial Railway Record No. 52 - p170-173*

The major advantage of good (robust) staging was the ability to accommodate steam-powered crainage. The largest machines are usually the block-setting machines, often known as Titans (originally non-revolving); Hercules (revolving or radiating); Mammoths (two very large machines used at Tynemouth. In reviewing the use of machinery to an ICE meeting in 1893, Pitt notes that the name Titan had been applied to all these, including revolving machines, see example for Peterhead in Figure 4.2 designed by W. Matthews for a working load of 50t, proved to 62.5t, and operated to a radius up to 30m. Most of those types of crane operated from the works being constructed. The travelling Goliaths used at Dover (see Figure 4.5) were essentially lighter and simpler using a small hoist to lift, and a traveller to move back and forth along the travelling beam. A smaller slewing crane is used to feed blocks to the Goliath. Similar Goliaths were using in the blockyard to produce and

**Figure 4.5 Dover - placing blockwork from travelling cranes**
*Courtesy Dover Harbour Board*

[20] Davies (1983) claims that the frightened horse in Figure 4.3 was uninjured!

store blocks in advance of construction. Lifting capacities at Dover were typically 40t, 42t or 60t.

In discussing the use of plant, discussers to Pitt (1893) made various recommendations. Wheels supporting the larger types of cranes should be sprung (not rigidly mounted) to limit damage to the track rails. Noting that many blockyards use gravity to move loaded wagons around, he observed the need for robust and reliable brakes. Similarly, Stothert (also of Stothert & Pitt) noted that much of the work of block-placing cranes was in controlling the descent of the blocks, particularly emphasising the importance of effective brakes on the winding barrel.

A summary of plant and methods available in (or before 1874) is given in Figure 4.6 from Cay (1874) on Aberdeen. We see pile installation using a small slewing crane, and blockwork installation from a travelling Goliath, all carried on staging. These operations are assisted by diving works supported by small dive boats – discussed in 4.2 and illustrated in Figure 4.6 below.

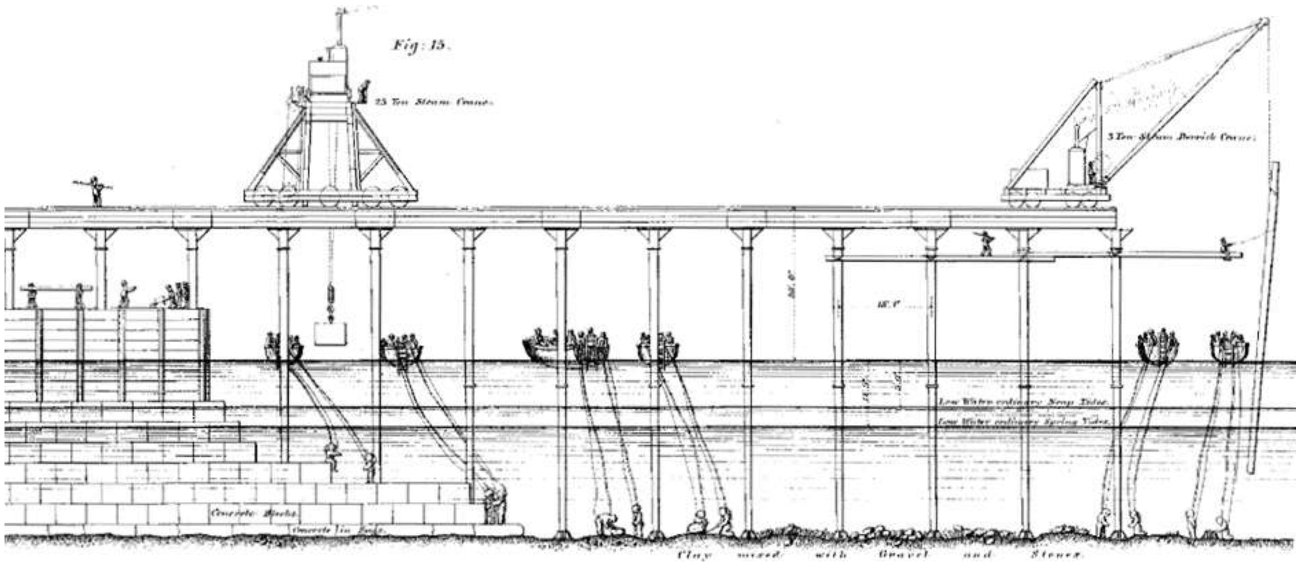



**Figure 4.6 Construction activities showing diver operations at Aberdeen**
*After Cay (1874)*

## 4.2 Underwater construction

### 4.2.1 Use of diving bells

Diving bells have been used in breakwater construction for many centuries, indeed it is possible that the Greeks or Romans used them (Franco & Verdesi, 1993). In the context of this thesis, diving bells have certainly been used around the UK since the late 1600s. The crucial improvement came when compressed air could be pumped into the bell, equalising the water pressure and prolonging the use, often ascribed to Smeaton who certainly used a diving bell of his own design at Ramsgate in 1788 (Skempton, 1981). Diving bells were used around 1812 by Rennie to form the pierheads at Howth (Cox & Gould, 1998) to level foundations for later block-setting. Another was used (again) by Rennie at Ramsgate in 1813.

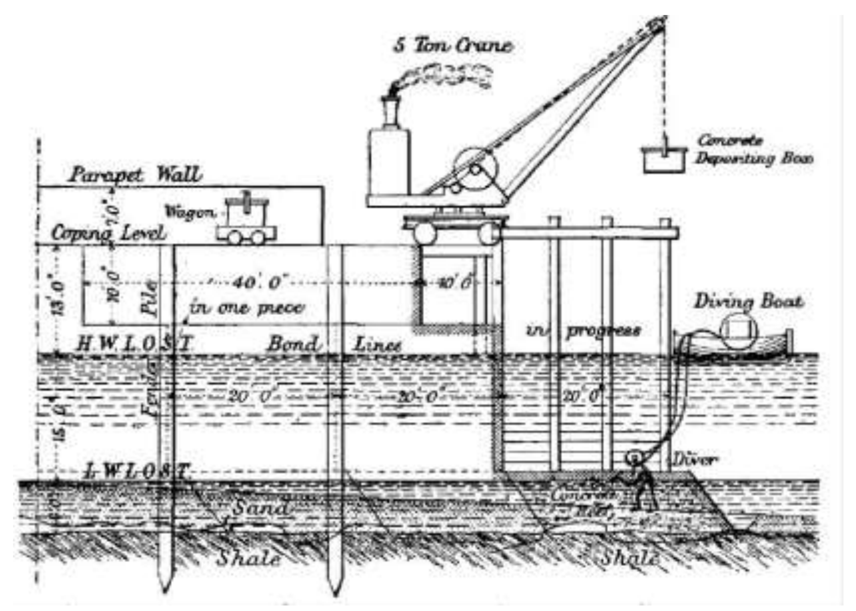



**Figure 4.7 Construction using divers and steam crane**
*From Kidd (1899)*

## 4.2.2 Use of helmet divers

Whilst diving bells were particularly useful in preparing the natural seabed and/or levelling foundation materials, the major advance in improving construction quality was the increasing availability of helmet divers. Together with use of block-setting cranes, use of divers permitted the placement of foundation blocks lower in the water column than previously, thus further away from disturbing forces as waves reflected from the wall. The net effect, as illustrated so well at Dover, was to allow the deletion of any mound or berm that could 'trip' the wave, substantially reducing the chances of breaking waves impacting on the wall, see methods to predict wave breaking in Chapter 5.

## 4.2.3 Floating construction plant

Earlier discussions on construction at Cherbourg and Plymouth identified the large number of (small) vessels using in many construction projects, principally moved by sail / wind, or by man-powered oars, or by winches. The developments of iron in making ships; and of steam power to move vessels saw the increasing use of steam powered vessels, and of some specialised equipment.

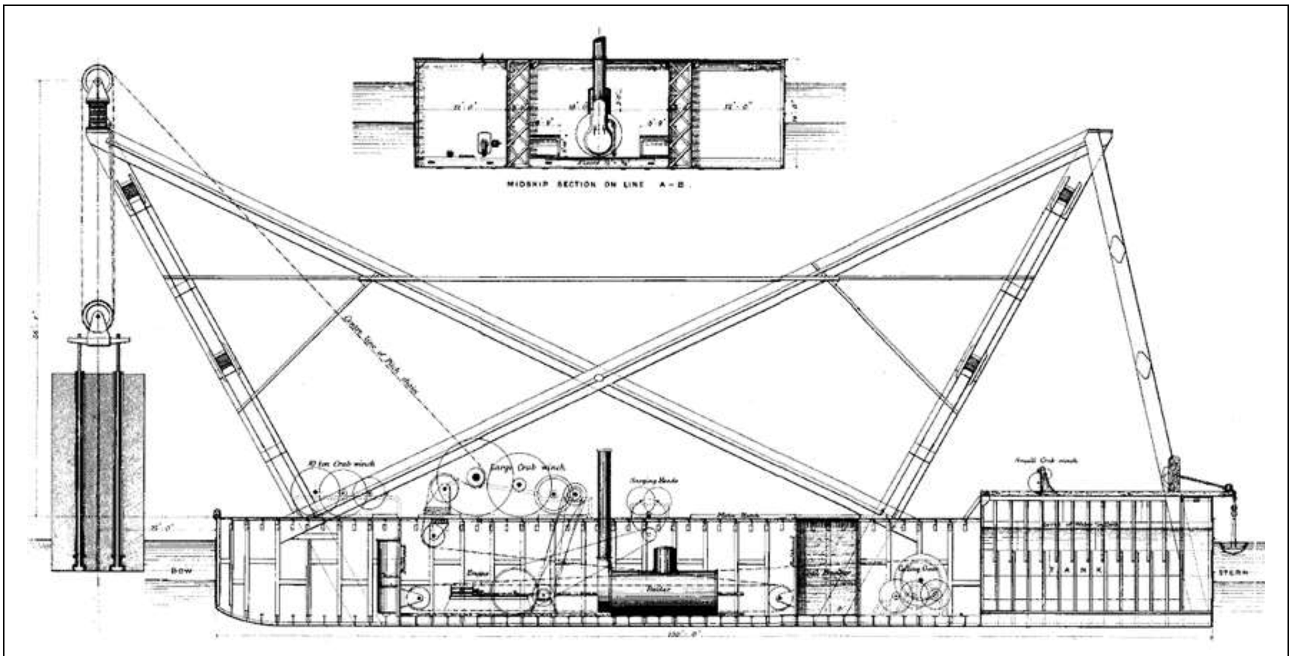



**Figure 4.8 Block-laying barge by Stoney (1874)**

The mobile sheerleg barge used by Stoney at Dublin is illustrated in Figure 4.8 in which we see that the steam power is used for crane and winch functions, but not for propulsion.

In contrast, the special bottom-dumping barge discussed by Carey (1886) shown in Figure 4.9 is driven by a propellor powered by the main steam boiler. This barge was used at Newhaven to place concrete in bags up to 100t.

Both of these vessels were highly specialised, being configured initially for a single task, although one can imagine both being adapted and re-used once their original purpose had been fulfilled.

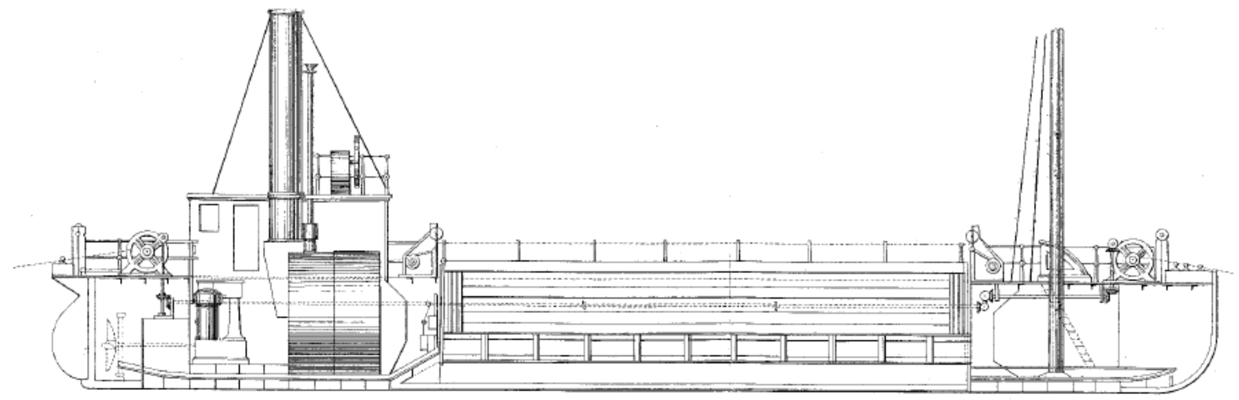


**Figure 4.9 Bottom-dumping barge for concrete – filled bags**
*After Carey (1886)*

## 4.3 Concrete mixing and material handling

Developments in the use of concrete blocks above required the development of concrete mixers, and machinery to handle input materials and the output concrete. Many of the advances in cranes have been covered in section 4.1, but so far little has been mentioned on concrete mixers.

Discussions following the paper by Scott (1858) in section 11.7 were mostly focussed on the form of breakwaters, but mentioned in passing an early example of a pier at Rye formed fully by 10t concrete blocks. Initially all concrete was mixed by hand, but in section 11.10 Stoney (1874) describes fabrication and placement of 350t concrete blocks with a 3HP steam engine used to power the mixers. In the following discussions (section 11.11), Parkes notes use of Messent mixers at Kurachee (probably developed around 1865) to make 27t blocks.

In reviewing use of construction plant, Vernon-Harcourt (1885) reviewed in 11.12 notes use of steam-powered cranes at Alderney (probably shortly before 1860, of Titan cranes at Ymuiden, Columbo, and Mormugao, and of a Goliath at Tynemouth in 1862. Sheild (1895) in 11.13 describes use of concrete by British engineers since ~ 1840, then cites mixers by Coode, Messent, Ridley, Le Mesurier, and Carey-Latham. A common 20-yard mixer usually delivered 12m$^3$ per hour. In 14.2, Pitt (1893) identifies concrete mixers by Messent, Lee, Ridley, and Punchard. In the discussion to Pitt's paper, Charles Walker notes that many Messent mixers were more than 20 years old, confirming their availability from (say) mid-1860s.

## 4.4 Summary of the advances

As might be expected, progress in improving the design and construction of these types of breakwaters was incremental, and the effects of many advances were inter-related. Development (and commercial production) of Portland Cement concrete from circa 1850 was accelerated by increased use of steam power to move materials (and wet concrete), and of steam cranes to place concrete blocks, through 1860-1870. Whilst concrete was initially mixed by hand, from ~1865 increasing use of steam engines powered a wide range of concrete mixers from circa 1870, leading to the wide-spread use of pre-cast concrete blocks.

Blockwork placement was originally based on over-end construction with block placement cranes running directly on the previously placed blockwork. The adoption of timber staging at Alderney and St Catherine’s from 1847 probably marked an important step in reducing wave and weather risks to the construction process, and speeding construction progress.

The objective of lowering the wall foundation away from the most aggressive area of wave breaking was substantially assisted by various methods of underwater working, primarily the availability of helmet divers, from about 1840, see sections 4.2 and 13.3.

Construction progress was further accelerated by the development of steam-powered vessels, see 13.5, and the use of methods to improve placement of underwater concrete, again see 4.2, and 13.2.

# 5 ANALYSIS TOOLS

This chapter identifies empirical analysis tools used to calculate indicative Factors of Safety for the breakwater types of interest in this thesis. These tools may be considered under three stages of analysis:

a) Wave analysis – takes 'offshore' wave conditions given by significant wave height and mean (or peak) wave period, and describes the simple methods then applied to the case studies in Chapter 6 to transform those wave conditions to the toe of the breakwater / wall being analysed.

b) Wave loads – calculates horizontal wave loads, and perhaps up-lift forces, for the main vertical (or steeply battered) walls being analysed in Chapter 6.

c) Factor of Safety calculations requiring a simple safety analysis using the wave loads above compared with sliding or overturning resistances calculated from simple statics.

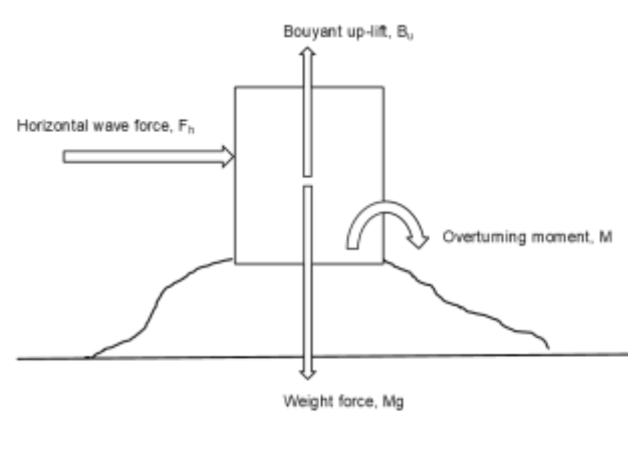



**Figure 5.1 Simple schematic of actions**

## 5.1 Outline of the safety analysis

The intention of the stability analysis here is to calculate Factors of Safety against sliding or overturning for representative sections of each breakwater studied. The primary drivers for these actions / responses are wave loads onto the breakwater wall, Figure 5.1, so the first steps in each case study have been to determine appropriate wave conditions 'offshore', then at the structure, from which are then calculated the intensity (%) of wave breaking onto the breakwater wall. The disturbing loads are then contrasted with the structure weight, frictional resistance, and restraining moments about the wall heel giving the sliding or overturning strengths. The most important wave loads are quasi-static momentum-driven loads, taken as acting sufficiently slowly to generate 'static' responses. Where appropriate, impulsive loads due to waves breaking onto the wall are also calculated.

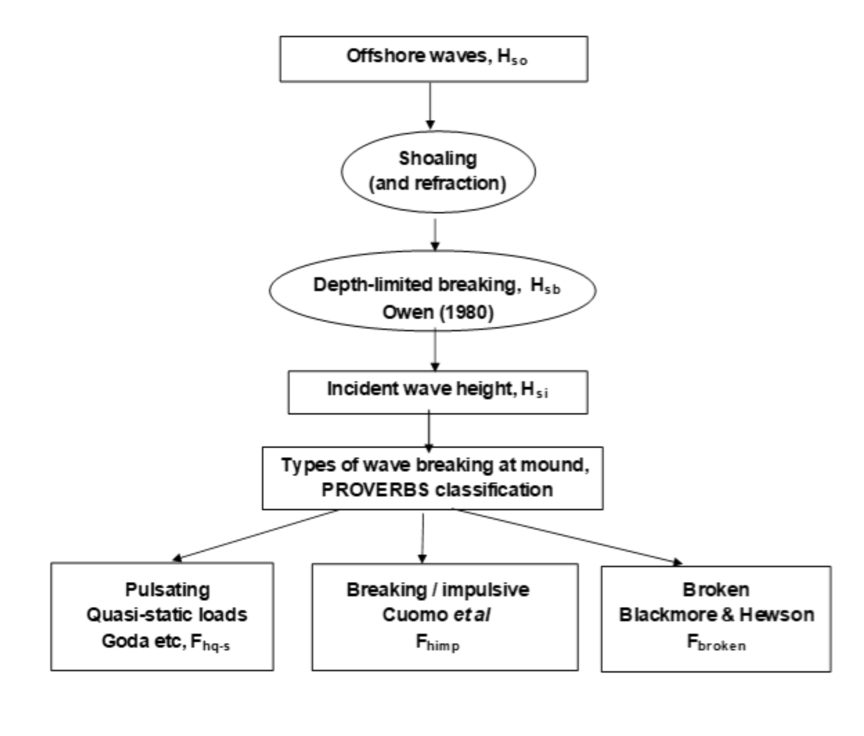



**Figure 5.2 Wave analysis steps used for case studies**

This analysis has intentionally used empirical models of general application. It is noted that site-specific analysis of a specific structure should however use physical model tests to give higher confidence in wave loads, and thus to validate or optimise any particular design, see e.g. general guidance by Hughes (1993), Wolters *et al* (2007, 2009).

## 5.2 Wave analysis

The general analysis of waves through to loads in the case studies discussed in Chapter 4, is illustrated in Figure 5.2. The first step in this analysis was to obtain estimates of 'offshore' wave conditions from data derived previously in (site-specific) wave hindcasting studies. In subsequent stages wave heights were shoaled, and then tested for wave breaking using a method that includes the effects of steeper bed slopes to account for natural shoals, and/or artificial mounds, section 5.2.2. The immediate precursor to calculating wave loads is to decide the form and degree of wave breaking onto the breakwater wall using the PROVERBS classification, discussed in section 5.3.

It will be noted that most wave effects are significantly influenced by local depths, so therefore by the water level. This is not fixed. The choice of water level to be used in analysis, indeed of the levels to be used, will depend on the response being calculated, the local tidal and surge contributions, and on the return period being considered. In general, higher water levels will permit larger depth-limited wave heights. But the role of local depth in causing local wave breaking against the wall is not so simple, so it is possible that some intermediate water levels may cause the worst wave breaking loads. (This certainly occurs at Alderney where wave loads are generally highest at mid-tide levels.) In the case studies in Chapter 6, wave loads have often been calculated at a number of different water levels.

### 5.2.1 Wave shoaling and refraction

For the case studies at Wick, Alderney and Dover discussed in Chapter 6, 'offshore' wave conditions have been derived from previous (site specific) reports. Those wave conditions were then checked for effects of shoaling and refraction. The breakwaters in Chapter 6 were either parallel to the main depth-contours (Wick and Alderney) or the depths are sufficient for refraction to be of only moderate consequence, so normal wave direction ($\beta$=0°) can be taken as the pessimistic case. At Dover, input wave conditions had already been refracted, but in most of the cases considered here, refraction effects on wave heights may be taken as negligible in comparison with depth-limiting. Shoaling may however still increase wave heights near to the breakwater, particularly over any steep mound, be-it natural or man-made. Two simple approaches have been used here: direct calculation of shoaling coefficient, below, or the simple nomogram for shoaling and refraction in Figure 5.3.

At its simplest, the effect of shoaling on wave height is applied using a shoaling coefficient $K_s$ multiplied by the offshore wave height, $H_o$, where:

$$K_s = \sqrt{((1/2n) \cdot (c_0/c))}$$

where: $c_0/c = \sin \alpha_0 / \sin \alpha$ with: $\alpha$ and $\alpha_0$ representing the angles between incoming wave crest and depth contour line for celerities $c$ and $c_0$. (Subscript zero refers to deep water conditions.) See definition sketch in Figure 5.3.

and where $n$ is defined:

$$n = \left[ 1 + \frac{\frac{4\pi h}{L}}{\sinh \frac{4\pi h}{L}} \right]$$

A simple approach to checking these calculations is to apply the nomogram in Figure 5.3, calculating $h/gT^2$ for the 'offshore' position (labelled here as $d/gT^2$). The combined refraction and shoaling coefficients ($K_r \cdot K_s$), and local (refracted) angle ($\alpha$) to the bed contour can then be read off the graph given $h/gT^2$ and the initial 'offshore' angle $\alpha_0$. Values of the shoaling coefficient alone ($K_s$) are read off the bottom axis, where $\alpha_0 = 0°$. These are then used to calculate the shoaled significant wave height, $H_{ss}$.[21]

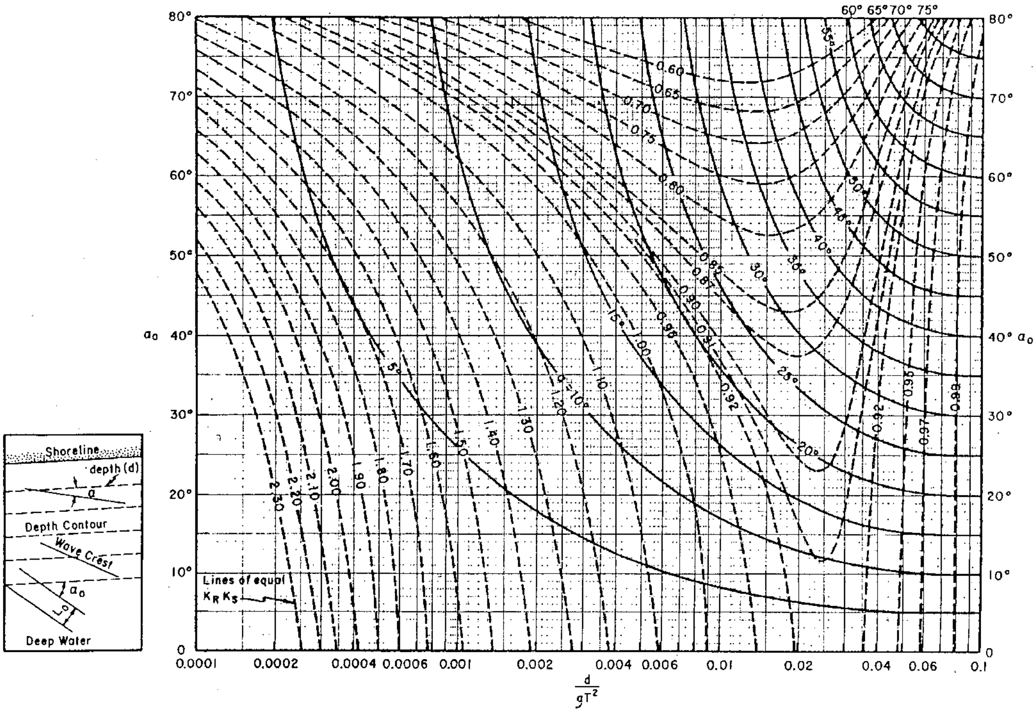



**Figure 5.3 Simple wave refraction and shoaling**
*From USACE (1977)*

### 5.2.2 Depth-limited breaking

The great majority of coastal structures, including most of the breakwaters considered in this thesis, are in intermediate or shallow water where the design wave heights may be broken before the wall by depth-limiting. The next stage in calculating waves at the breakwaters is therefore to check whether, and to what extent, wave heights might be reduced by breaking. This can be critical for breakwaters formed on natural shoals (as at Wick) or on rubble mounds (as at Alderney). Depth-limited wave heights may be approximated by simple relationships

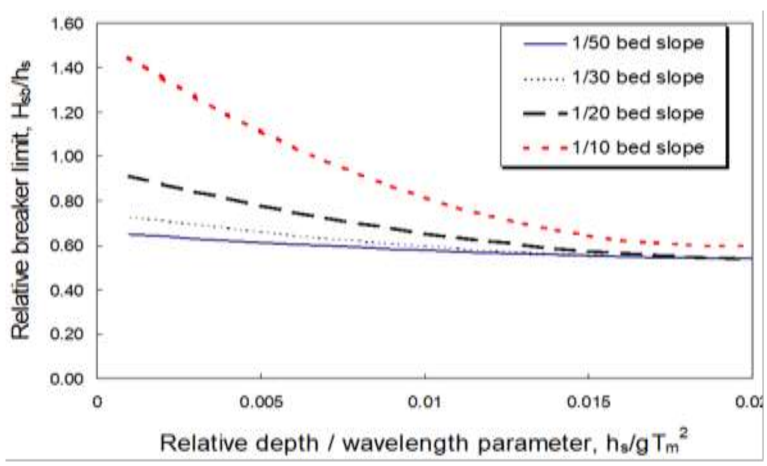



**Figure 5.4 Depth limited wave breaking**
*After method of Owen (1980)*

[21] A cautionary note on use of $\alpha$ and $\beta$ as wave angles. In 'modern' coastal engineering nomenclature, wave obliquities have been defined by $\beta$ whereas $\alpha$ has been used as slope angle. Unfortunately, the methods discussed here date from before those (generally European) agreements, so use 'old' definitions. There may also be similar confusions over the symbols for depth, $h$ or $d$. It is therefore important to check definitions for condition context.

between the limiting breaking (significant) wave height $H_{sb}$, the local water depth $h_s$, and the local relative water depth, $d^* = (h_s/ gT_m^2)$, but these do need to take account of bed slope. For initial analysis, the very simple relationship between local water depth ($h_s$) and breaking significant wave height ($H_{sb}$) may be assumed, but only for bed slopes of 1:50 or flatter:

$$H_{sb} = 0.55h_s$$

The important aspect of the Owen (1980) curves in Figure 5.4 is that they include the effects of steep bed slopes ($m_f$) that are often over-simplified (or ignored) by other methods. Various relationships have been suggested, (most biased towards or validated for shallow bed slopes, flatter than 1:30) but there is not yet an established universal design method, although that proposed by Goda (1985, 2000) is most widely accepted.

For steep slopes, the simple limit of $H_{sb} = 0.55h_s$ under-estimates breaking wave heights, perhaps substantially, so Owen (1980) derived curves in Figure 5.4, to which empirical equations in Table 5.1 below were fitted:

**Table 5.1 Breaker limits, after Owen (1980)**

| **Bed Slope** | **Breaking limit, $H_{sb}/h_s$** |
|---|---|
| 1/100 | $H_{sb}/h_s = 0.58 - 2\ (h_s/ gT_m^2)$ |
| 1/50 | $H_{sb}/h_s = 0.66 - 10.6\ (h_s/ gT_m^2). + 229\ (h_s/ gT_m^2)^2$ |
| 1/30 | $H_{sb}/h_s = 0.75 - 20.1\ (h_s/ gT_m^2) + 480\ (h_s/ gT_m^2)^2$ |
| 1/20 | $H_{sb}/h_s = 0.95 - 38\ (h_s/ gT_m^2) + 896\ (h_s/ gT_m^2)^2$ |
| 1/10 | $H_{sb}/h_s = 1.54 - 98\ (h_s/ gT_m^2) + 2540\ (h_s/ gT_m^2)^2$ |

These calculations give estimates of the breaking limit $H_{sb}$. It is important to note that this is NOT necessarily the incident wave height. In use, the breaking limit should be compared with the shoaled incoming wave height ($H_{ss}$). The incident wave height at the breakwater, $H_{si}$ will be the smaller of $H_{ss}$ and $H_{sb}$. This then serves as input to the following stages of calculations.

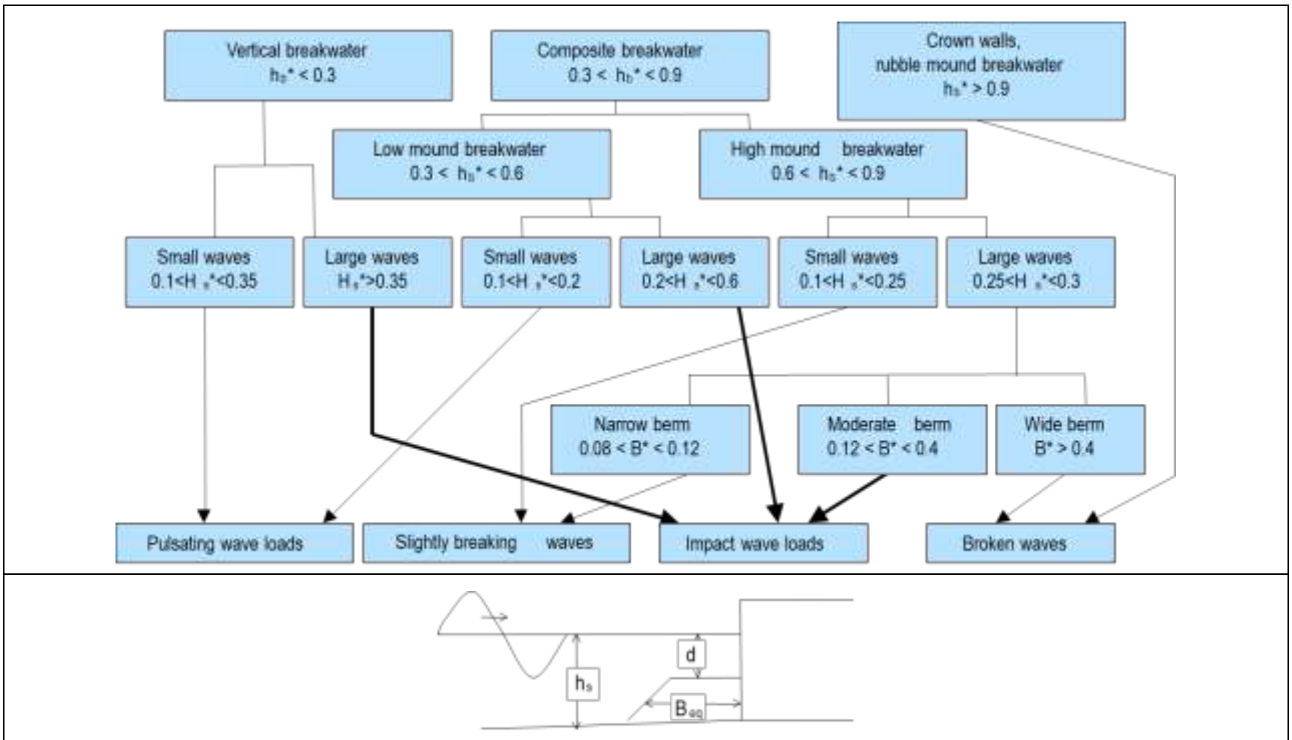



**Figure 5.5 PROVERBS parameter map to predict type of wave loading**
*After Oumeraci et al (2001)*

## 5.3 Occurrence of wave force regimes

Having identified the incident wave conditions, the procedures here then have to identify the types of wave loadings as in the simplified parameter map Figure 5.5 developed in the PROVERBS research project reported by Oumeraci *et al* (2001) based on physical model measurements, Allsop *et al* (1995, 1996).

The 'rules' in this diagram identify conditions where impulsive loads are likely to occur by assessing three dimensionless parameters:

$h_b* = h_b/h_s$ mound or berm height relative to total water depth;

$H_s* = H_{si}/h_s$ incident wave height relative to water depth;

$B* = B_{eq}/L_p$ mound or berm width relative to wavelength.

The limits given in Figure 5.5 indicate the effect of a breakwater mound on the nature of likely wave loads on the wall. These calculations may often be repeated for a number of different water levels.

Examples of pulsating (non-impulsive) versus impulsive breaking are illustrated in Figure 5.6 photographed from wave flume tests at Edinburgh after Bruce & Van der Meer (2016).

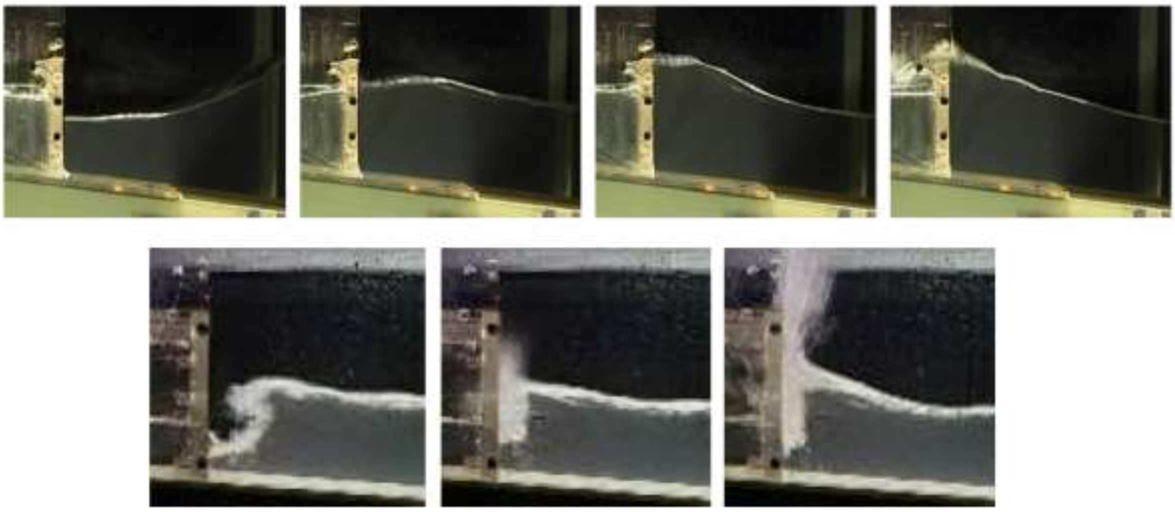


**Figure 5.6 Forms of wave breaking onto vertical walls from EurOtop (2017)**

*Upper - Pulsating waves giving quasi-static loads and 'green water' overtopping;*
*Lower - Impulsive breaking onto wall giving impulsive loads and overtopping.*

A simple model of wave breaking approaching the structure was developed within PROVERBS by Calabrese to give estimates of the proportion or % of wave impacts on vertical / composite walls. Breaking occurs when, at the structure, the incident wave height with exceedance probability of 0.4%, ($H_{99.6}$) is higher than a critical breaker height $H_{bc}$, defined as a transition wave height between breaking and non-breaking in front of the structure. The equivalent berm width, $B_{eq}$, is estimated at ½ berm height. The peak wavelength $L_p$ is the wavelength associated with the peak period $T_p$ in the local water depth $d_s=h_s$.

The critical wave height at breaking, $H_{bc}$, is defined for depths where breaking occurred in tests by Allsop *et al* (1996), $0.07 < h_s/L_p < 0.25$:

$$H_{bc} = (0.1025 + 0.0217\ C^*)\ L_p \tanh (2\pi\ k_b\ h_s/L_p)$$

where:

$$k_b = 0.0076 \cdot (B_{eq}/d)^2 - 0.1402 \cdot (B_{eq}/d) + 1 \text{ for } 0 \le B_{eq}/d < 10$$

and:

$C^* = (1 - C_r) / 1 + C_r)$

The reflection coefficient $C_r$ from these types of walls may be estimated using guidance by Allsop (1995):

For simple vertical walls and small mounds $C_r = 0.95$

For low-crest walls, $(0.5 < R_c/H_{si} < 1.0)$ $C_r = 0.8 + 0.1\ R_c/H_{si}$

For composite walls, large mounds, heavy breaking $C_r = 0.5$ *to* $0.7$

The uncertainties in predicting wave breaking on a mound point to a conservative approach assuming $C_r = 1$, so $C^* = 0$, then $H_{bc}$ reduces to:

$H_{bc} = 0.1025\ L_p \tanh (2\pi\ k_b\ h_s/L_p)$

The incident wave height, $H_{si}$, is compared with $H_{bc}$ to give categories of breaking:

$H_{si} / H_{bc} \leq 0.6$ No evident breaking and wave load is pulsating

$0.6 < H_{si} / H_{bc} < 1.2$ Wave breaking occurs and may give impacts

$H_{si} / H_{bc} \geq 1.2$ Heavy breaking, may give broken wave loads

The next step is to estimate % of breaking waves $P_{b\%}$:

$P_{b\%} = \exp[-2\cdot(H_{bc}/H_{si})^2]\cdot 100\%$

For $H_{si}/H_{bc} \geq [H_{si}/H_{bc}]_{bro}$ some waves will arrive already broken, so these should be subtracted from the % of breaking waves to give potential impacts $P_{i\%}$ on the structure, estimated:

$P_{i\%} = \{\exp[-2\cdot(H_{bc}/H_{si})^2] - 0.58\cdot\exp[-1.93\cdot(H_{bc}/H_{si})^2]\}\cdot 100\%$

Values of $P_{i\%}$ may then be used to re-appraise the likely loading case:

$P_{i\%} < 2\%$ ⇒ Little breaking, wave loads are primarily pulsating

$2\% < P_{i\%} < 10\%$ ⇒ Breaking waves give impacts

$P_{i\%} > 10\%$ ⇒ Heavy breaking give impacts or broken loads

## 5.4 Magnitude of wave forces

The first step in estimating wave loads is to estimate the slowly-acting (often termed 'quasi-static') momentum driven loads. The most widely used prediction method for wave forces on vertical walls was developed by Goda (1974, 1985, 2000), calculating horizontal forces for caissons on rubble mound foundations. But before considering Goda's method, it is often useful to review the very simple methods by Ito and/or Hiroi, see Ito (1971), Goda (1985).

Hiroi's formula calculates a uniform wave pressure ($p_{av}$) on the front face up to $1.25H$ above still water level:

$p_{av} = 1.5\rho_w gH$

where $H$ is assumed to be $H_{max}$. Ito uses Hiroi's formula where the relative water depth over the mound, $d / H_s < 2$, and Sainflou's methods when $d / H_s > 2$. Sainflou's (1928) method generally gives $p_{av} = 0.8$ to $1.0\rho_w gH$, lower than Hiroi's.

Ito's method is taken to give a rectangular distribution of pressures on the front face, calculated in terms of $H_{max}$, determined for two different regions of relative water depth, $H_{max}/d$, where $d$ = depth at the toe of wall:

*for* $H_{max}/d < 1$ $p_{av} = 0.7\ \rho_w gH_{max}$

*for* $H_{maxdt} > 1$ $\quad\quad\quad$ $p_{av} = \rho_w g H_{max} (0.15 + 0.55\, H_{max}/d)$

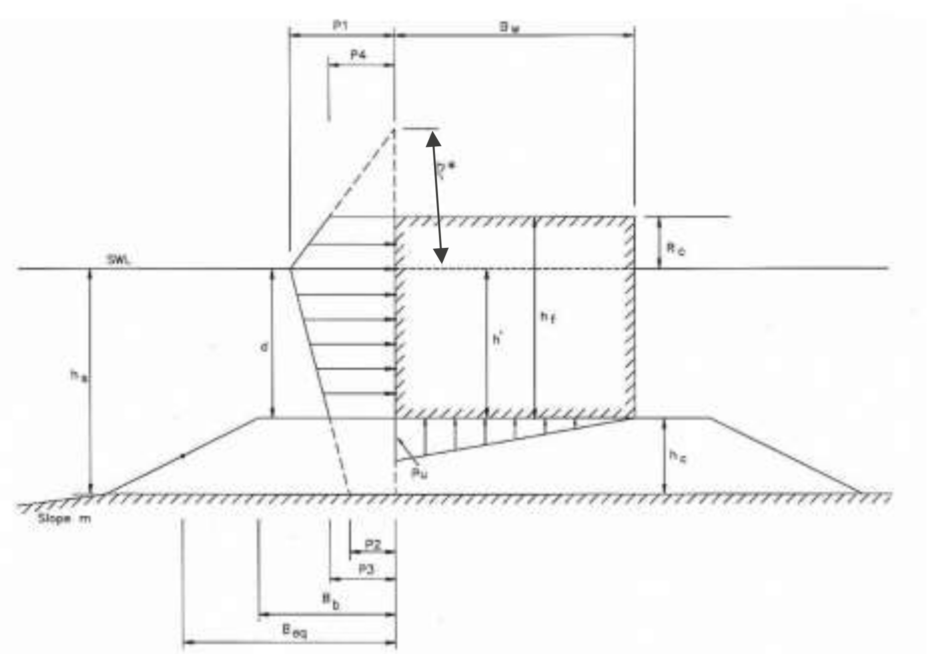



**Figure 5.7 Notation used for Goda's equations for wave force**
*Adapted, after Goda (2000)*

The more complete (and widely accepted) wave force prediction method by Goda (1974, 1985, 2000) was calibrated against laboratory tests on sliding caissons and analysis of historic caisson breakwater failures. The method represents wave pressures on the wall by a trapezoidal distribution, reducing from $p_1$ at the static water level (s.w.l.) to $p_3$ at the caisson base (Figure 5.7). Above s.w.l., pressures reduce to zero at the notional run-up point given by $\eta^*$ above s.w.l giving $p_4$ at the wall crest.

Underneath the caisson, up-lift pressures at the seaward edge ($p_u$) are determined by a separate expression. Up-lift pressures are distributed triangularly from the seaward edge to zero at the rear heel.

Goda emphasised that this method did not purport to predict wave pressures *per se*. In practice, many researchers have found that it does give good estimates of pressures for non-impulsive conditions. Goda's method is widely accepted as giving the best estimate of total momentum-driven forces, even if it is a little complicated to apply.

The main parameters determined in Goda's method are:

$$\eta^* = 0.75(1+\cos\beta)H_{max}$$

$$p_1 = 0.5(1+\cos\beta)(\alpha_1+\alpha_2\cos^2\beta)\rho_w g H_{max}$$

$$p_2 = p_1 / (\cosh(2\pi h/L))$$

$$p_3 = \alpha_3 p_1$$

$$p_u = 0.5(1+\cos\beta)(\alpha_1\alpha_3)\rho_w g H_{max}$$

Coefficients $\alpha_1$, $\alpha_2$, and $\alpha_3$ are determined from:

$$\alpha_1 = 0.6 + 0.5\,[\,(4\pi h/L)/\sinh(4\pi h/L)\,]^2$$

$$\alpha_2 = \min\{\,((h_b-d)/3h_b)(H_{max}/d)^2\,,\ 2d/H_{max}\,\}$$

$\alpha_3 = 1 - (h'/h) [ 1 - 1/\cosh(2\pi h/L) ]$

Where $\eta^*$ is the maximum elevation above s.w.l. to which pressure could be exerted (taken by Goda as $\eta^* = 1.5H_{max}$ for normal wave incidence); $\beta$ is the angle of wave obliquity. The wave height, $H_{max} = 1.8H_s$ seaward of the surf zone, but in broken waves, $H_{max} = H_{maxb}$. The depth $h=h_s$ is taken at the toe of the mound, and $d$ over the mound at the front face of the caisson, but $h_b$ is taken $5H_s$ seaward of the structure, not shown in Figure 5.7 as it is further seaward.

The total horizontal force, $F_h$, (at 1/250 exceedance) is calculated by integrating pressures $p_1$, $p_2$, and $p_3$ over the height $h_f$ of the front face, see Figure 5.7. Total up-lift force is calculated by integrating from $p = p_u$ at the front edge to $p = 0$ at the rearward edge, giving a total up-lift force $F_u = 0.5\ p_u\ B_c$.

The 'Goda forces' calculated above have been shown to be the most reliable representation of momentum-driven loadings, and Allsop (2000) recommends that these loads should always be assessed even if the earlier analysis suggest that impulsive loads may be important.

## 5.5 Impulsive wave forces

### 5.5.1 Force magnitudes and durations

Impulsive wave forces may be much greater in magnitude than the quasi-static loads discussed above, but are of far shorter duration, and therefore unable to generate the response of a load of longer duration. The example in Figure 5.8 shows relative pressures, $p/\rho gH_s$. The lower (rounded) traces show typical quasi-static or pulsating loads. The short spikes show impulsive loads, in this instance up to $p/\rho gH_s \leq 20$. In the experiments at Wallingford by Allsop *et al* (1996) and McKenna (1997), peak impulsive pressures reached $p/\rho gH_s \leq 40$.

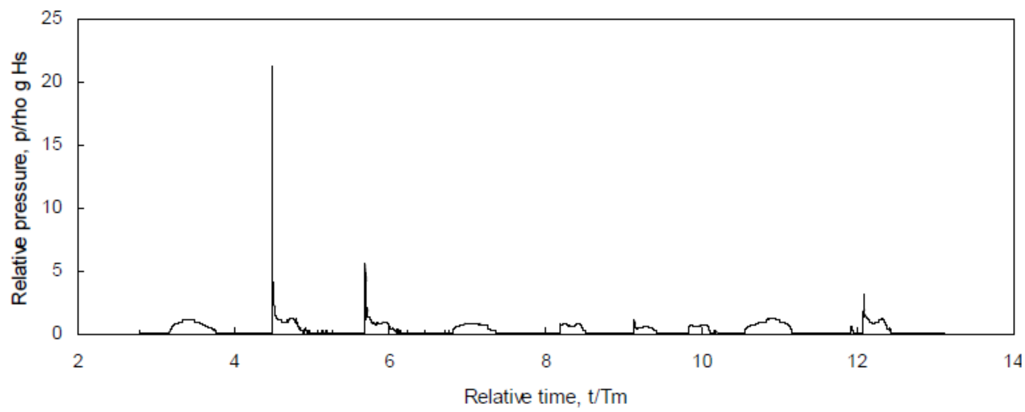



**Figure 5.8 Examples of pulsating and impulsive forces**
*After Allsop et al (1996)*

Various formulae have been developed to give estimates of short-duration impulsive loads. The magnitudes of impulsive loads are strongly influenced by relative mound level, primarily depth over the mound, $d$. Based on moderate scale tests at Wallingford and large scale tests in the large wave flume at UPC Barcelona, Cuomo *et al* (2010) developed two simple formulae for impact ($F_{h,imp}$) and quasi-static ($F_{h,qs}$) forces at 1/250 level, given by:

$$F_{h,imp} = C_r^{1.65} \cdot \rho \cdot g \cdot H_{si} L_s \cdot (1-(h_b - d) / d)$$

$$F_{h,qs} = 4.8\ \rho\ g \cdot H_{si}^2$$

where again the reflection coefficient $C_r$ may be estimated using the guidance by Allsop (1995), the wavelength $L_s$ is determined at the structure toe in depth $d$, and $h_b$ is the water depth at breaking, usually evaluated 5 wave heights seaward of the wall.

Taken alone, impulsive forces are however of little significance, as their effect on any structure depends strongly on the dynamic response characteristics of the receiving structure, here the breakwater wall. Limiting impulsive forces may be related to impulse duration (usually given by the rise time, $t_r$) by a simple inverse power relationship. From large-scale data, Cuomo *et al* (2010) suggest:

$$F_{imp} = 7\, t_r^{-0.6}$$

This relationship was first shown by Cuomo *et al* (2010) using dimensioned data. A more generic relationship was then developed by Cuomo *et al* (2011) where relative horizontal forces ($F_{imp}/F_{qs}$) were compared against dimensionless rise time ($t_r/T_m$), Figure 5.9.

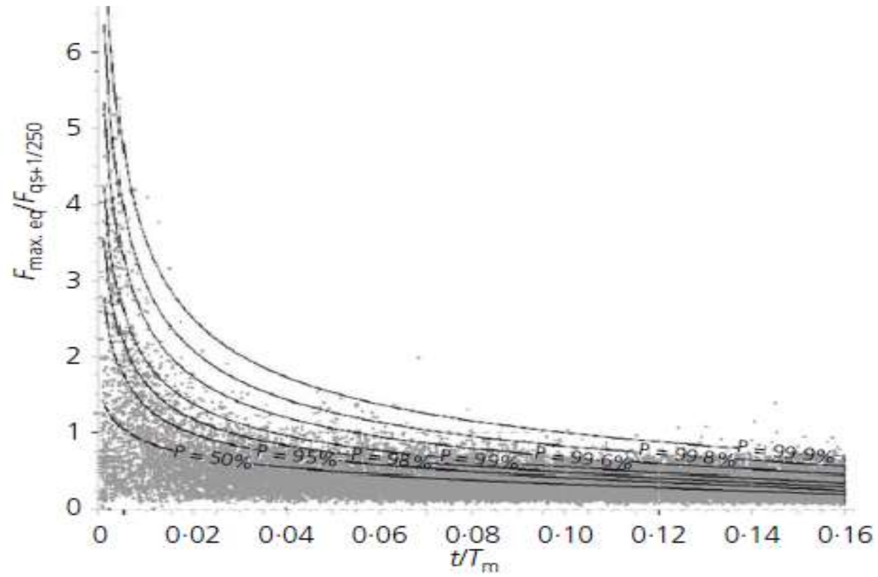



**Figure 5.9 Relative impulsive force $F_{imp}/F_{q\text{-}s}$ versus relative rise time $t_r/T_m$**
*After Cuomo et al (2011), contours show non-exceedance levels*

Field data discussed by Bullock *et al* (2005) supported the relationship $p_{max}$=3.1/$t_r$, proposed by Blackmore & Hewson (1984).

An important implication of Figure 5.9 and related graphs is that any combination of impulsive force ($F_{imp}/F_{q\text{-}s}$) and rise time ($t_r/T_m$) lying along any curve of given non-exceedance level (say 99.9%) are equally likely. The further implication of this is that any value of $F_{imp}$ (or $P_{imp}$ for pressures) are simply one of many possible forces (or pressures). Whether any particular force (or pressure) is exceeded is simply a matter of chance influenced by the structure geometry, and the wave breaker form and aeration, themselves influenced by the breaking of previous waves. This goes to explain in part why different authors have found substantially different impulsive forces (or pressures).

A further complication is given by the degree of aeration. This is discussed in part by Blackmore & Hewson (1984) for broken waves, see section 5.6 below, but this method does not apply to the impulsive breaking discussed here. One of the sole methods to take account of air content was described by Partensky (1988) based on tests in the large wave channel at Hannover, GWK, where impact pressures of 0.01 to 0.03s may be calculated from:

$p_{dyn} = K_L \rho g H_b$

where $H_b$ is the breaking wave height, and $K_L$ is given in terms of the air content $a_e$ of the breaking wave:

$K_L = 5.4 ( (1/a_e) - 1)$

There are however almost no measurements of aeration in waves breaking onto vertical (or battered) walls, although this was an objective of research which deployed pressure / aeration units (PAU) on Alderney breakwater, and on a large scale test section in the large wave flume (Grosser Wellenkanal or GWK) at Hannover, see Bullock *et al* (2007). Bullock *et al* (2005) who show a single graph of measured void ratios, but it is notable that for all the discussion on aeration and its effects, measured values are extremely sparse.

### 5.5.2 Spatial coherence

Impulsive loads are not only limited in duration, they are also limited in the space over which they apply. It is always difficult to estimate the area / length over which impulsive forces apply as there are very few measurement data. Following the PROVERBS project, Allsop (2000) made estimates of "least-bad" spatial correlation widths / length for impact forces on long walls or caissons under oblique / short-crested waves, summarised

- For heavy impacts ($F_{Impact}/F_{Goda}$ >> 2.5), and small obliquity or spreading, assume a typical coherence width ≤ $L/16$;
- For light impacts ($F_{Impact}/F_{Goda}$ < 2), normal wave attack ( $\beta = 0°$) and little spreading, assume a typical coherence width ≤ $L/4$;

where the coherence width (or length) is a length along the structure in which the impulsive load is sensibly consistent, so may be assumed as constant. For impulsive loads, it is expected that the load will reduce significantly outside of this space.

## 5.6 Broken wave forces

For many coastal seawalls, and for some breakwaters, the design wave condition may be limited by depth in front of the structure, and the main wave load is given by these broken waves. The larger waves at the structure will be broken, see parameter map previously in Figure 5.5, and it is most unlikely that wave loads will be impulsive. A method to estimate an average wave pressure from broken wave loads was developed by Blackmore & Hewson (1984).

$p_{i\ max} = \lambda_a \ \rho T_p C_b^2$

where $\lambda_a$ is an aeration coefficient for which values are suggested in Table 5.2, $\rho$ is the water density, $T_p$ is the wave spectral peak period, and $C_b$ is the velocity of the breaker at the wall. The simplest formula for breaker celerity may be given by shallow water wave theory:

$C_b = (gd)^{1/2}$

These methods may be used to make an initial estimate of the horizontal wave force under broken waves, $F_{hBroken}$:, to be applied only if $F_{hBroken} < F_{hGoda}$:

$F_{hBroken} = h_f \cdot p_{i\ max} = h_f \lambda_\alpha \rho T_p C_b^2$

**Table 5.2 Aeration coefficients, $\lambda_a$ for broken wave loads (after Blackmore & Hewson, 1984)**

| Bed slope | 1:5 to 1:10 | 1:30 to 1:50 | 1:100 |
|---|---|---|---|
| Foreshore conditions | | | |
| Smooth bed, sand | 1.5 | 0.9 | 0.7 |
| Rough, rocky | 0.5 | 0.3 | 0.24 |
| Very rough, emergent rocks | 0.13 | 0.18 | 0.14 |

## 5.7 Stability analysis, sliding and overturning

During this period, breakwaters, even the breakwater walls, were seldom constructed monolithically. If however the blocks and any fill were well bonded or cemented, then the superstructure may often have performed as if monolithic. It is convenient therefore to start any stability analysis by assuming that the breakwater superstructure behaves structurally as if it were a single unit.

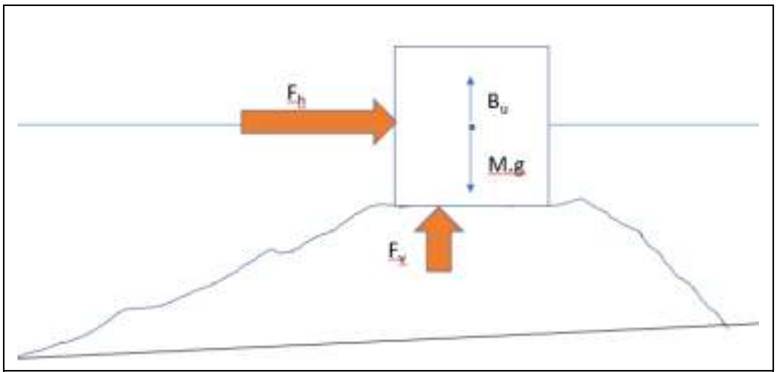



**Figure 5.10 Main loadings for stability analysis**

Extending the simplified loading diagram in Figure 5.10, we can identify two applied loads, $F_h$ and $F_v$, together with self-weight $M.g$, and the buoyant uplift $B_u$., summarised in Figure 5.11.

### 5.7.1 Sliding analysis

In any sliding analysis, the applied horizontal load ($F_h$) is balanced against the net restraining force of friction between superstructure and rubble ( = $(M.g - F_v - B_u) . \mu$. Sliding occurs if:

$$F_h > (M.g - F_v - B_u) . \mu$$

The Factor of Safety against sliding, $FoS_{slide}$ is given by:

$$FoS_{slide} = ( (M.g - F_v - B_u) . \mu) / F_h$$

Values of the friction coefficient have been derived by various authors, see discussion by Stevenson (1878) in section 11.8, and by Hutchinson *et al* (2010), and are commonly assumed to be in the region $\mu \approx 0.7$ to 0.8. In this analysis, sliding failure will certainly occur for $FoS_{slide} < 1$.

### 5.7.2 Overturning analysis

Overturning analysis is slightly more complicated, requiring an assumption of a rotation point, see Figure 5.11.

At the first stage this is often assumed to be around the rear heel of the superstructure, and this is probably correct for the onset of any overturning, although any loss of bearing capacity at that point will move the effective rotation point forward, thus reducing both the restraining moment, and the contribution to the disturbing moment from $F_v$.

Maintaining the simplified analysis, lever arms for the applied loads, $F_h$ and $F_v$, can be defined by the positions relative the rear heel of those loads as $x_h$ and $x_v$ respectively. The weight and buoyancy will each act at ½ the superstructure width, $B_w$.

The disturbing moment is then: $= F_h \,.\, x_h + F_v .\, x_v + B_u \,.\, B_w/2$

The restoring moment is: $(M.g.\ B_w/2)$

The Factor of Safety against overturning, $FoS_{over}$ is then given by:

$$FoS_{over} = (M.g \,.\, B_w /2)\ /\ (F_h \,.\, x_h + F_v .\, x_v + B_u \,.\, (B_w/2))$$

Overturning failure will certainly occur for $FoS_{over} < 1$.

For a simple concrete caisson on a rubble mound, the uplift force $F_v$ can generally be calculated by Goda's method, see section 5.4. For the types of structure analysed here, it is not however clear that any or all of the dynamic up-lift will act on the superstructure, so $F_v$ might be reduced in the stability analysis accordingly, giving a more optimistic result.

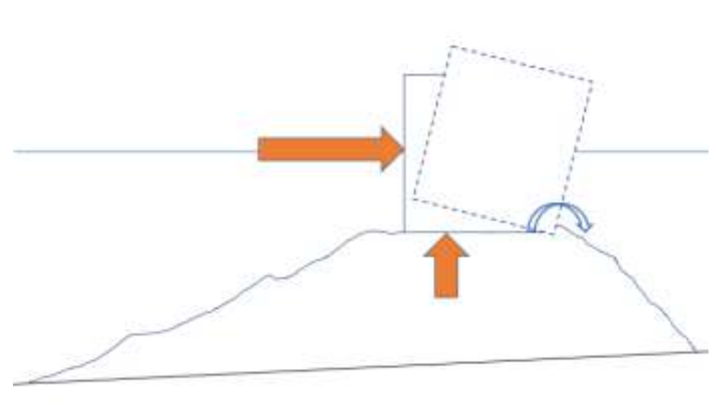


**Figure 5.11 Schematic for overturning analysis**

## 5.8 Blockwork failure

As has been seen earlier in this thesis, many of the structures of interest were far from monolithic, and failures often occur by loss of individual blocks, see also Chapter 7.

One aspect of wall stability that has been discussed in Chapter 3 is the potential for incipient overturning to progressively reduce the inter-block pinching forces in the front face. It might therefore be possible to balance a hypothetical outward force from section 3.5 on selected blocks against an inter-block weight force reduced in proportion to the net overturning moment.

We saw from section 3.5 that short-duration internal pressures might reach order of 0.25 x the impulsive pressure acting on the outer face. Now in many instances, impulsive pressures may commonly reach 2-5 times the quasi-static load, see section 5.5.1. The consequence is that pressures of the order of the quasi-static wave loads might apply to the rear-side of any loose block.

A major remaining 'gap' is that there are however no measurements of blockwork restraint, although Muller and co-workers did illustrate in Figure 3.9 possible approaches using photo-elastic materials to identify instances of zero down-ward pinching forces. Taking this approach further might therefore require systematic modelling beyond the capacity of the current research.

# 6 APPLICATION of ANALYSIS METHODS to SELECTED STRUCTURES

## 6.1 General approach and selection of case studies

As part of the analysis of ‘old breakwaters’, the stability of three example breakwaters from those reviewed in Chapter 2 has been analysed using analytical methods developed over the last twenty years by the author and co-researchers. This is illustrated by the three case studies in this chapter:

- Wick (designed by Thomas Stevenson, failed 1870-77);
- Alderney (damaged even during construction, lost its outer length 1865-89);
- Dover (still shows high stability after 110 years).

In these case studies, representative cross-sections have been derived from historical records, as have the approach bathymetry. Representative wave conditions are transformed to the breakwater toes, including depth-limiting and impulsive breaking effects. Empirical formulae developed during and since the PROVERBS project have been used to explore occurrence of impulsive loads, and then the magnitudes of the main momentum and impulsive loads. Factors of Safety (*FoS*) against sliding and/or overturning have been determined for each example structure studied here over a range of representative wave conditions. That analysis is then compared with reality for each case study to identify whether relatively simple empirical methods can be used to give realistic estimates of a structure’s safety level.

The general approach to deriving wave conditions has been described earlier in Chapter 3. In each case, representative wave conditions have been derived from historic wave analysis studies at HR Wallingford. At both Wick and Alderney, the approach bathymetry and/or the rubble mound will have had substantial effects on the wave condition, including shoaling and wave-breaking. Waves at Dover have been extracted from a recent HR Wallingford study using a refraction point close to the outer breakwaters.

The methods to calculate shoaling and depth-limiting to incoming waves applied here were discussed in Chapter 3.

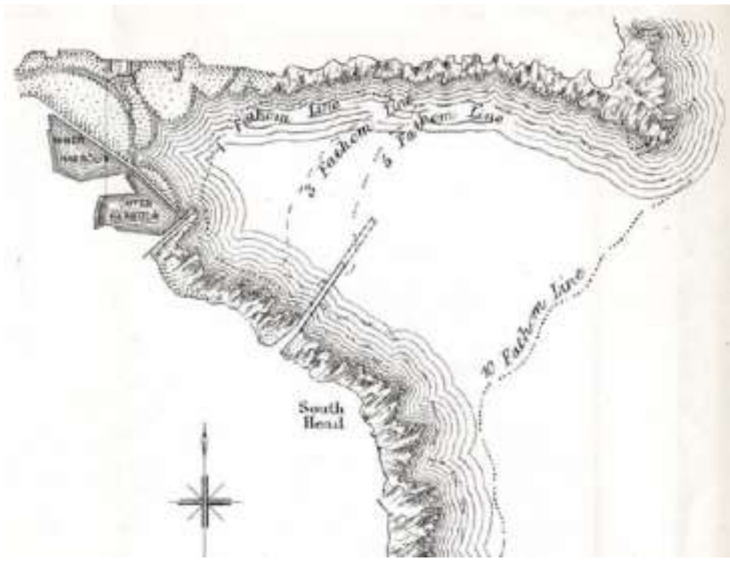
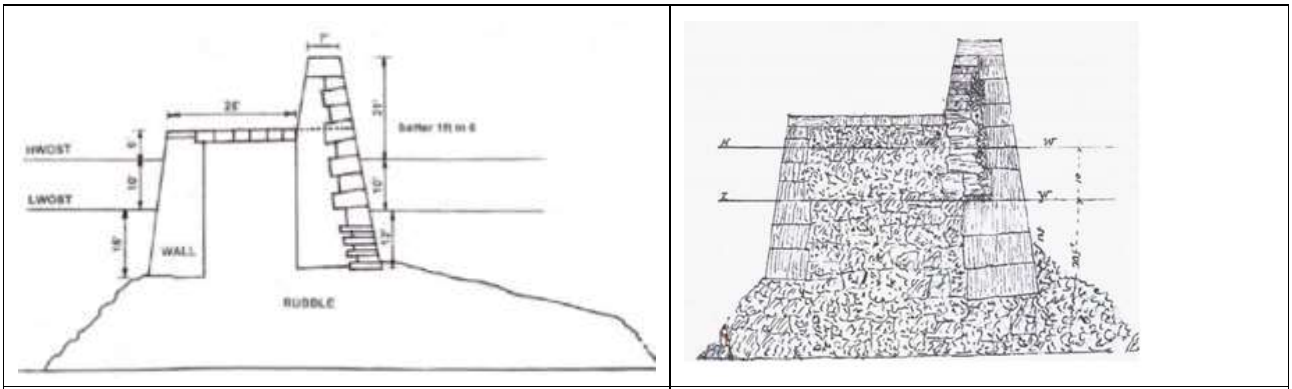



**Figure 6.1 Proposed new breakwater in Wick Bay**
*From Vernon-Harcourt (1885)*

## 6.2 Case study 1, Wick

### 6.2.1 Outline of the structure

Wick harbour in north-east Scotland, discussed in Chapter 2, was a major fishing harbour in the 1700 and 1800s, expanded in 1811, and again in 1825-1834. Even so, by 1857 further capacity was needed so the British Fishery Society proposed a new breakwater to provide an enlarged (outer) harbour. Plans and specification for an expanded harbour, Figure 6.1, were drawn up by D. & T. Stevenson in 1862, see Paxton (2009). Construction of Stevenson’s breakwater began in April 1863 to a planned length of 460m.




| Design section ~ 270m from root | Wall section at 320m from root |
|---|---|

**Figure 6.2 Cross-sections of Stevenson's breakwater**
*From Paxton (2009)*

The design by Stevenson (1874) was a rubble mound to -5.5mLW following the Crane Rocks surmounted by block walls (Figure 6.1), filled between by rubble, with a superstructure width up to 16m. The seaward wall was formed as slice-work battered at 6:1 (Figure 6.2). Below water, blocks were dry-jointed, but above high water used Roman cement at the start of the works, then Portland cement mortar. Paxton claims that the depth to which blocks were taken at -5.5mLW was lower than usual, probably to avoid impulsive breaking on the mound, and/or to reduce the likelihood of movement of foundation material.

By October 1867, the rubble mound had reached 326m, breakwater walls to 250m. By September 1868, the completed breakwater had reached 320m (Figure 6.3), but in October the outer 75m was demolished by wave action with the wall down to -4.6 m LW. Paxton (2009) records that in February 1870, the outer 116m was knocked over to -1.8 mLW (above wall foundation level) in a storm estimated at $H_{max} \approx 12.8$m, so $H_s \approx 7$m.[22]

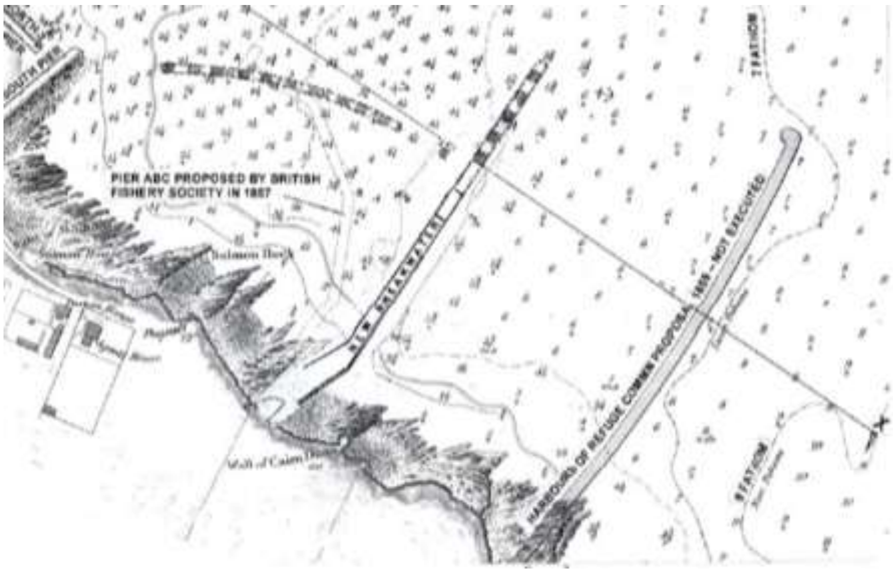



**Figure 6.3 Location of Stevenson's breakwater**
*From Paxton (2009)*

[22] Paxton probably based this on the unpublished ICE paper by Doull, see Paxton, 2009 for details.

In the summer of 1870, the outer 55m length was rebuilt (shortened to 260m instead of the original 480m). The parapet was omitted (assumed to reduce wave forces, and particularly any overturning). The end was stepped (again probably to reduce forces). The top was rebuilt in concrete over coursed blockwork. At the vertical end, large concrete blocks were tied together by iron bars to give a degree of monolothicity.

In December 1872, the replacement composite end (1372 t) was demolished down to -3mLW, being "*slewed round by successive strokes until removed*". Paxton (quoting an unpublished ICE paper by Doull) notes that "*the rubble base (*at -5mLW*) is said to be not much disturbed*". A further 46m "*of solid structure set in cement*" were then destroyed with water 8-9m deep passing over the parapet. After further damage in January 1877 when a 2642t end was destroyed in storms, Stevenson abandoned the project in August 1877.

The failure history by Paxton (2009) suggests both impulsive and non-impulsive wave loads. Additional loads will probably have been imparted by overtopping down-fall pressures, see Bruce *et al* (2001), Wolters *et al* (2005). The Safety Factor analysis below will start with momentum-driven loads, later extending to include impulsive loads.

### 6.2.2 Derivation of representative wave conditions

The general bathymetry of Wick Bay is shown here in Figure 6.4 adapted from Admiralty chart 1462 by Hydraulics Research Station. It shows the shoal of the Crane Rocks along which the Stevenson breakwater was built. Paxton (2009) shows the layout along the 5-fathom (9.1m) contour, together with an aborted 'harbour of refuge' breakwater following the 7-fathom (12.8m) contour. A red box has been overlain on Figure 6.4 indicating the approximate position of the Stevenson breakwater. This position has been used to estimate seabed levels for wave transformation calculations in the wave analysis.

The tidal range at Wick is 3.8m. The main tidal levels (relative to LAT=CD) are listed by Paxton (2009) from Admiralty Tide Tables:

| | | | |
|---|---|---|---|
| MHWS | +3.5mCD | MHWN | +2.8mCD |
| MSL | +2.0mCD | MLWN | +1.4mCD |
| MLWS | +0.7mCD | | |

Paxton (2009) suggests that Stevenson "*would have expected waves of ($H_s$=) 7-9m*". For their model tests in 1975, Hydraulics Research Station (HRS) derived a 1:1 year condition as $H_{max}$=12m, ($H_s$=6.7m) and a 1:50 year as $H_{max}$=18m, ($H_s$=10m). Wave periods used were $T$=14s for the longest fetches down to $T$=7s for frequently occurring conditions. Stability calculations here have used $H_s$=8m and $H_s$=10m.

To derive incident conditions, offshore waves must be transformed to account for shoaling and breaking over the last 50-100m to the breakwater wall. Three representative sections have been taken across the line of Stevenson's breakwater mound (box in Figure 6.3), approximately normal to the -10mCD contour, and taken at chainages of 100m (A), 180m (B) and 250m (C) from the shoreline. Seabed slopes (including the shoal) average 1:10-1:20 at about 50m from the breakwater toe, at: -3mCD for section A; -5.4mCD for B; and -7mCD for C.

The central estimate of any water level during a storm should be mean water, so here around MSL=+2mCD. During any large storm, it is probable that a storm surge will elevate water levels, perhaps by 1-2m suggesting a mean value of 1.5m. These calculations have therefore been run for the (nominal) water level of 3.5mCD, conveniently MHWS.

Three alternative wave periods ($T_p$ = 10, 12, and 14s), two nominal wave heights of $H_s$= 8 and 10m, and a water level of +3.5mCD, have been used to estimate wave heights for each of mound section lines: A (100m from root), B (180m from root), and C (250m from root).

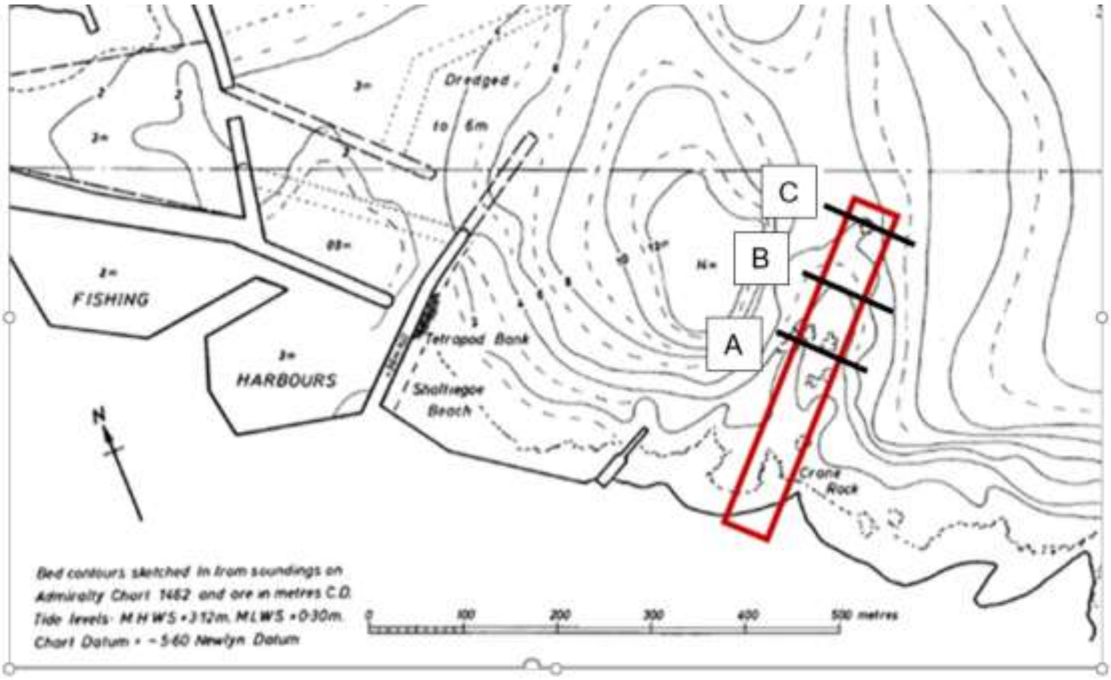



**Figure 6.4 Bathymetry of Wick Bay**

*After Hydraulics Research Station (1975), (outline of breakwater (red box) and section lines are added)*

**Table 6.1 Results of wave transformation calculations, Wick**

| Mound section | From shore | $H_s$ | $T_p$ | Wave steep | Bed level | Water depth | Bed | $K_s$ | $H_{ss}$ | $H_{sb}$ | $H_{si}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | m | m | s | - | mCD | m | 1:x | | m | m | m |
| A | 100 | 10 | 14 | 0.033 | -3 | 6.5 | 10 | 1.2 | 12.0 | 8.2 | 8.2 |
| B | 180 | 10 | 14 | 0.033 | -5.4 | 8.9 | 10 | 1.1 | 11.2 | 10.6 | 10.6 |
| C | 250 | 10 | 14 | 0.033 | -7 | 10.5 | 10 | 1.1 | 10.8 | 12.0 | 10.8 |
| | | | | | | | | | | | |
| A | 100 | 10 | 12 | 0.044 | -3 | 6.5 | 10 | 1.1 | 11.3 | 7.6 | 7.6 |
| B | 180 | 10 | 12 | 0.044 | -5.4 | 8.9 | 10 | 1.1 | 10.5 | 9.5 | 10.5 |
| C | 250 | 10 | 12 | 0.044 | -7 | 10.5 | 10 | 1.0 | 10.2 | 10.5 | 10.2 |
| | | | | | | | | | | | |
| A | 100 | 10 | 10 | 0.064 | -3 | 6.5 | 10 | 1.1 | 10.5 | 6.6 | 6.6 |
| B | 180 | 10 | 10 | 0.064 | -5.4 | 8.9 | 10 | 1.0 | 9.9 | 7.9 | 7.9 |
| C | 250 | 10 | 10 | 0.064 | -7 | 10.5 | 10 | 1.0 | 9.6 | 8.5 | 8.5 |
| | | | | | | | | | | | |
| A | 100 | 8 | 12 | 0.036 | -3 | 6.5 | 10 | 1.1 | 9.0 | 7.6 | 7.6 |
| B | 180 | 8 | 12 | 0.036 | -5.4 | 8.9 | 10 | 1.1 | 8.4 | 9.5 | 8.4 |
| C | 250 | 8 | 12 | 0.036 | -7 | 10.5 | 10 | 1.0 | 8.2 | 10.5 | 8.2 |
| | | | | | | | | | | | |
| A | 100 | 8 | 10 | 0.051 | -3 | 6.5 | 10 | 1.1 | 8.4 | 6.6 | 6.6 |
| B | 180 | 8 | 10 | 0.051 | -5.4 | 8.9 | 10 | 1.0 | 7.9 | 7.9 | 7.9 |
| C | 250 | 8 | 10 | 0.051 | -7 | 10.5 | 10 | 1.0 | 7.7 | 8.5 | 7.7 |

In the first stage of the wave analysis, classic linear shoaling (discussed previously in Chapter 3) has been used to estimate shoaling coefficients, $K_s$ (column 8 in Table 6.1). Those values of $K_s$ have been checked (with good agreement) using the SPM graphical method (see Figure 5.3 in Chapter 5). Shoaled (significant) wave heights in column 9 in Table 6.1 ($H_{ss}$), are then tested in column 10 for depth-limited breaking using the methods by Owen (1980) also discussed in Chapter 5, taking account of the approach bed slopes (1:10) for sections A, B and C) to give $H_{sb}$. Lastly, the lesser of $H_{ss}$ and $H_{sb}$ are listed as the incident wave height, $H_{si}$ in column 11 of Table 6.1.

We see immediately the influence of these steep bed-slopes. Shoaling is substantial (~20% increase) for longer wave periods ($T_p$ = 12s or 14s). On the steep bed slopes, breaking limits are substantially higher than they would be for shallow slopes. Working seaward, wave heights at mound section A are always depth-limited, but they shoal and break later for mound section B. Any condition where $H_{sb}$ is less than $H_{ss}$ indicates significant breaking. Waves are substantially larger at the outer end (mound sections B and C, 180m and 250m from the shoreline).

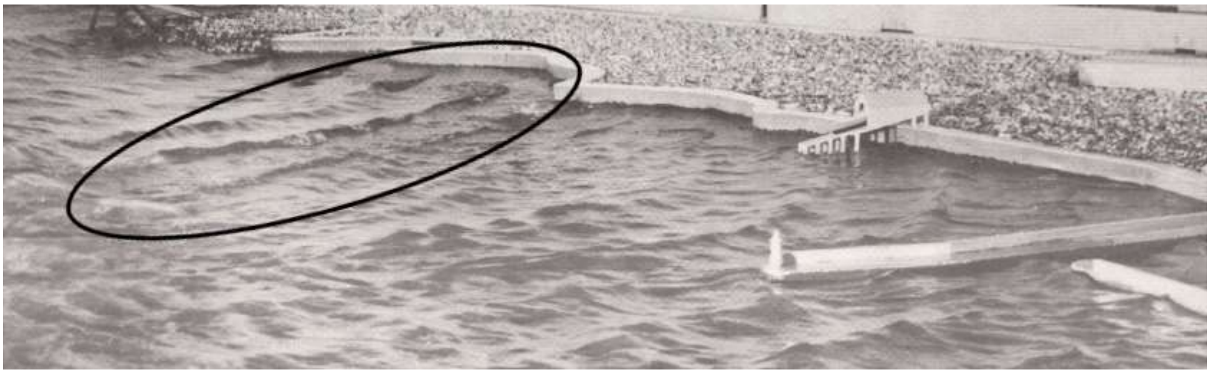


**Figure 6.5 Physical model showing enhanced shoaling and breaking at Crane Rocks**
*From HRS (1975), 8s waves*

Qualitative confirmation of these processes is illustrated by photographs of the Hydraulics Research Station model (HRS, 1975). Figure 6.5 shows significant shoaling (and then breaking) over the Crane Rocks on which Stevenson had built his breakwater.

From the calculations in Table 6.1, we see that longer wave periods increase shoaling and depth-limited heights. Most focus on the wave load calculations that follow will therefore be given to $T_p$ = 10s for $H_s$ = 8m as a best-guess storm condition, and $T_p$ = 12s with $H_s$ = 10m as an upper estimate test condition.

### 6.2.3 Breakwater section

Paxton (2009) shows two wall sections, here in Figure 6.2, at about 270-320m from the shoreline, but they require interpretation before calculating wave loads. Firstly, levels in Paxton's sections need to be related to Chart Datum (CD). It is assumed here that HWOST = MHWS = +3.5mCD. The walkway deck is at +6.4mCD, parapet crest at +9.9mCD, wall toe on the mound at -3.2mCD and seabed about -8mCD. The structure width at LW is about 13m, and the mean parapet width 2.7m.

For a wall foundation level at -3.2mCD, the breakwater wall is 9.6m high and 13m wide. The 2.7m wide parapet adds a further 3.5m height locally. The parapet density is assumed at 2.6t/m$^3$, being fitted stone blocks and concrete. The lower section will be less dense as the fill may be (say) 1.8t/m$^3$. Taken overall, the dry weight of this section will be 270t/m. (The submerged weight will be less, depending on water level.)

The toe levels are slightly at variance with Paxton's 'failure record' which has the wall founded at -3.6mLW in October 1864, so perhaps landward of analysis section C here. For October 1867, Paxton gives the wall founded at -5.5mLW at 250m out, probably close to analysis section C, (taken at 250m out from the shoreline).

For analysis of wave loads at C, two alternative wall sections were originally considered, based on dimensions derived from Figure 6.2, but with the wall foundation at either -3.2mCD (section C1), or at -5.5mCD (section C2). The (dry) weight of section C2 therefore increases by 54t/m to 303t/m.

### 6.2.4 Occurrence of breaking

The calculations of wave heights above show the effects of wave shoaling and depth-limited breaking, but they do not tell information on the form of wave breaking onto the breakwater wall. To do this the PROVERBS classification in Figure 5.5 in Chapter 5 can be most helpful.

Starting with position C at the outer end, local depths reach $h_s$ = 13m, and over the foundation mound, $d_b$=6.7m. So $h_b$* =$h_b/h_s$= 0.5, classed in the PROVERBS 'parameter map' as a 'low mound'.

Because approach slopes are steep at 1:10 to 20, it is not surprising that relative wave heights are high at $H_s$* =$H_s/h_s$=0.6 to 0.75, giving 'large waves', even for $H_s$~8m. The foundation mound is relatively small, $B^*=B_{eq}/L_p$ ~ 0.1-0.2, so classed as a 'moderate mound'.

Taken overall, these methods suggest that many wave loads on the breakwater wall will be impulsive (see 'Impact loads' box in Figure 5.5 in Chapter 5), particularly over the outer end where waves are shoaled by local bathymetry, as illustrated in Figure 6.5.

It is worth noting that $H_{bc}$ is a fictional rather than measured parameter and is used purely to discriminate the proportion of wave breaking. Values of $H_{bc}$ may therefore differ significantly from breaking wave heights determined by other methods.

**Table 6.2 Wave breaking calculations, Wick**

| Section | $H_s$ | $T_p$ | Bed level | Local depth | $H_{si}$ | $H_{bc}$ | $H_{si}/H_{bc}$ | $P_i$ | $P_b$ |
|---|---|---|---|---|---|---|---|---|---|
| | m | s | mCD | m | m | m | - | % | % |
| A, 100 | 10 | 12 | -3 | 6.7 | 7.6 | 2.84 | 2.7 | 76 | 31 |
| B, 180 | 10 | 12 | -5.4 | 9.5 | 10.5 | 4.47 | 2.3 | 70 | 29 |
| C, 250 | 10 | 12 | -7 | 11.5 | 10.2 | 5.61 | 1.8 | 55 | 22 |
| | | | | | | | | | |
| A, 100 | 8 | 10 | -3 | 6.7 | 6.6 | 2.80 | 2.4 | 70 | 29 |
| B, 180 | 8 | 10 | -5.4 | 9.5 | 7.9 | 4.37 | 1.8 | 54 | 22 |
| C, 250 | 8 | 10 | -7 | 11.5 | 7.7 | 5.43 | 1.4 | 37 | 15 |

The results of these calculations are summarised in Table 6.2. These confirm the view formed earlier, even at $H_s$=8m (and certainly at $H_s$=10m), that wave breaking onto the breakwater will have been substantial with breaking waves of order 15-30%.

### 6.2.5 Calculations of loads

Wave loads on the wall are influenced significantly by local wave heights, and by the mound geometry, so loads do not always respond in a simple way to different inputs. As discussed in Chapter 3, wave loads have been calculated using methods by both Ito (1971) and Goda (1985).

An initial observation from the load calculations in Table 6.3 is that Ito's simple method gives total forces that are higher than Goda's more complicated method, and probably therefore more conservative. (Ito pressures are often higher than those from Goda, but are only applied over the submerged wall height, so not including the wall to the higher of $R_c$ or $\eta^*$ as applied in Goda's method).

**Table 6.3 Wave load calculations, Ito and Goda, Wick**

| Section | $H_s$ | $T_p$ | $p_{av}$ | $F_{h(Ito)}$ | $\eta^*$ | $p_1$ | $p_2$ | $p_3$ | $F_{h(Goda)}$ | $F_{u(Goda)}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| | m | s | kN/m$^2$ | kN/m | m | kN/m$^2$ | kN/m$^2$ | kN/m$^2$ | kN/m | kN/m |
| A, 100 | 10 | 12 | 197 | 2583 | 21 | 137 | 125 | 125 | 1714 | 851 |
| B, 180 | 10 | 12 | 145 | 2313 | 28 | 192 | 179 | 174 | 2906 | 1187 |
| C, 250 | 10 | 12 | 185 | 3315 | 28 | 187 | 177 | 169 | 3192 | 1158 |
| | | | | | | | | | | |
| A, 100 | 8 | 10 | 150 | 1969 | 18 | 116 | 102 | 101 | 1427 | 710 |
| B, 180 | 8 | 10 | 136 | 2159 | 21 | 142 | 128 | 122 | 2100 | 859 |
| C, 250 | 8 | 10 | 111 | 1978 | 21 | 139 | 128 | 119 | 2314 | 840 |

In the main, the Goda forces are greater at offshore mound section C than inshore. Longer wave periods also increase wave loads, although that increase is greater at A (100m) or B (180m) than at C (250m).

### 6.2.6 Calculations of Factors of Safety

The simplest stability analysis is to compare horizontal and vertical loads versus sliding and overturning resistance given by weight and friction. This requires various simplifications, but the approach is generally robust.

The first stage in estimating sliding resistance is to compute the weight of a representative section. The wall section (Figure 6.2) can be divided into three parts:

- Rubble mound, approximately 34m wide at the base and from -9.6mCD up to the wall toe -3.2mCD, so about 6.4m high.
- Main wall section founded at -3.2mCD (ignoring the ~1m lower on the rear side), and rising to the walkway at +6.4mCD, so about 9.6m high. At mid-height, the wall is about 13m wide (front to back), so occupies about 125m$^2$.
- Parapet wall to +9.9mCD, so about 3.5m high, and of an average width of 2.7m, so occupying about 9.5m$^2$.

Loose fill will have a (dry) density of about 1.7t/m$^3$, but the density of fitted masonry will be closer to 2.7t/m$^3$. In the analysis of example breakwaters by Allsop *et al* (2017), the average ratio of block to fill were 30% to 70%. So for the main wall, the weight can be calculated as 30% x 2.7t/m$^3$ (blocks) and 70% x 1.7t/m$^3$ (fill). Adding in the parapet wall at 2.7t/m$^3$, the total weight reaches 265t/m. But that is dry weight, so we need to subtract buoyant up-lift of 87t/m at a water level of +3.5mCD, giving a net weight to resist sliding of 179t/m.

Sliding resistance is then computed in Table 6.4 by applying a friction coefficient of $\mu = 0.78$ (taken from full-scale friction tests by Hutchinson *et al*, 2010). Overturning moments are computed about the rear heel of the superstructure apportioning wave load and lever arms from the 'Goda' analysis, including buoyancy. Factors of Safety are plotted for $T_p$=10s in Table 6.4.

For a caisson breakwater, a final stage before computing sliding, or overturning resistance, would be to apply the wave-driven up-lift force, further reducing the restoring force and overturning moment. But here, the 'base' of the structure is permeable, so uplift pressures may not act together to lift the superstructure. Calculations of overturning and sliding in Table 6.4 have therefore not used $F_u$. This is an optimistic interpretation which will yield higher Factors of Safety than if the up-lift forces were to be included.

**Table 6.4 Results of Factors of Safety analysis (Wick)**

| Section | $H_s$ | $T_p$ | Bed level | $H_{si}$ | $F_h$ | $F_u$ | $B_u$ | Mg-$B_u$ | FoS | FoS |
|---|---|---|---|---|---|---|---|---|---|---|
| | m | s | mCD | m | kN/m | kN/m | kN/m | kNm | Sliding | Overturning |
| A, 100 | 10 | 12 | -3 | 7.6 | 1714 | 851 | 850 | 1753 | 0.8 | 0.8 |
| B, 180 | 10 | 12 | -5.4 | 9.5 | 2906 | 1187 | 1163 | 2051 | 0.6 | 0.6 |
| C, 250 | 10 | 12 | -7 | 10.2 | 3192 | 1158 | 1373 | 2244 | 0.6 | 0.5 |
| | | | | | | | | | | |
| A, 100 | 8 | 10 | -3 | 6.6 | 1427 | 710 | 850 | 1753 | 1.0 | 1.0 |
| B, 180 | 8 | 10 | -5.4 | 7.9 | 2100 | 859 | 1163 | 2051 | 0.8 | 0.8 |
| C, 250 | 8 | 10 | -7 | 7.7 | 2314 | 840 | 1373 | 2244 | 0.8 | 0.8 |

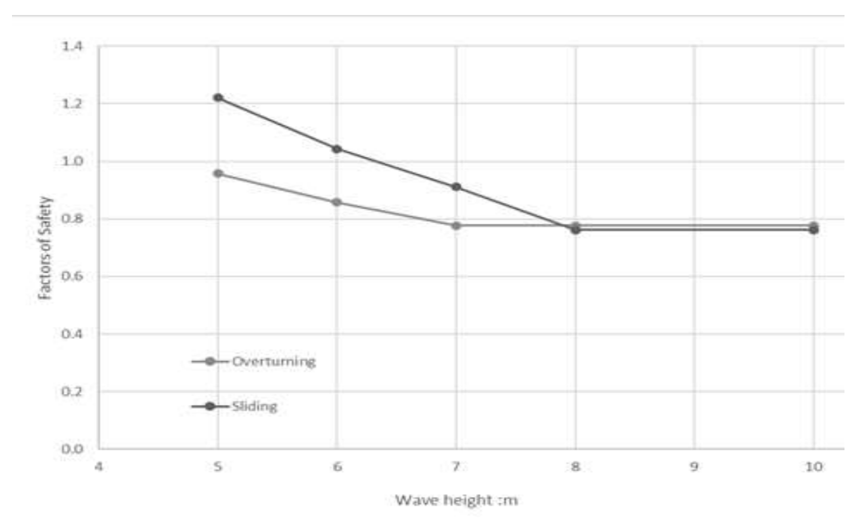



**Figure 6.6 Effect of wave height on Factors of Safety**
*Wick section B, $T_p$=10s*

Despite this optimistic assumption, both sliding and overturning calculations give Factors of Safety below unity, *i.e.* suggesting failure, for the conditions tested. At the inshore end, the breakwater wall reaches *FoS*=1 for the shorter wave periods, but all other conditions fall below *FoS*<1.

These calculations were then extended for lower wave heights, giving Factors of Safety for both sliding and overturning plotted in Figure 6.6. These suggest that Stevenson's breakwater would probably only have been stable (*FoS*>1) for waves up to $H_s$<4.7m (overturning) or $H_s$<6.3m (sliding). That the reduction of *FoS* for overturning is not steeper with reducing wave height may be the influence of buoyancy (fixed for a given water level), whereas wave forces reduce directly with reducing wave height.

### 6.2.7 Impulsive loads

It is almost superfluous to discuss impulsive loadings given that the calculations have already demonstrated that Stevenson's breakwater would have been unstable against quasi-static sliding and overturning for wave heights $H_s$<5.5m, even without including the effect of wave up-lift forces. We saw earlier however that the steep bed slopes and rock mound will have shoaled and/or broken incident waves, especially for wave periods, $T_p \geq$ 10s. Not surprisingly, the PROVERBS analysis by Calabrese's method to identify occurrence of breaking suggests that breaking is most severe for section A where $P_b \approx$ 29-33%, whilst further out at sections B and C, $P_b \approx$ 15-23%.

The method to estimate impulsive loads by Cuomo *et al* (2010) only gives meaningful results for section A where $F_{imp} / F_{Goda} \approx$ 1.2-2.0, with the greater impacts for the longer wave periods, $T_p \geq$12s. Impulsive loads are however of short duration. Consulting Figure 5.9 in Chapter 5 after Cuomo *et al* (2011), impulsive forces of 2 times quasi-static forces are associated at 99% probability with relative rise times, $t_r/T_m \approx$ 0.01, so of order 0.1s. Analysing the extent to which such impulsive loads would cause movement is beyond this analysis, requiring assessment of the dynamic characteristics of the wall, including dynamic characteristics of its foundation, and added mass of the water behind the wall that might be displaced. These dynamic loads will however have further reduced *FoS*, already below unity.

### 6.2.8 Conclusions to case study on Wick

Stevenson's breakwater at Wick was in a bay exposed to substantial storms from North, East and South-East. Even in the absence of validated wave prediction methods, Stevenson might well have expected waves of $H_s$= 7-9m. It may have been less clear that waves of $T_p$ ≈10-14s would shoal substantially over the Crane Rocks shoal on which the breakwater was built. Wave breaking would also have been delayed by the steep bed slopes so that incident wave heights might have been greater than expected relative to the local water depth. These effects would then have been aggravated by the rock mound on which the breakwater wall was placed.

Applying present day techniques to calculate local wave conditions demonstrates that the breakwater as built would not have survived without mobilising restraint beyond that apparent in this analysis, or some mechanism to abate wave forces.

For many such structures, movement of the foundation mound or scour will have reduced foundation support to the toe blocks, hence allowing the wall blocks to lose bonding. This failure mode may well have contributed to the failure at Wick, although there is no longer any evidence to allow its analysis. In any case, the global stability calculations here are enough to demonstrate the potential for global wall failure.

## 6.3 Case Study 2, Alderney

### 6.3.1 History of the structure

The Admiralty harbour on Alderney was one of two constructed in the Channel Islands as 'Harbours of Refuge'. The general history of these harbours of refuge was discussed in Chapter 2 and by Allsop (2020), and the development of Alderney harbour in section 2.5.

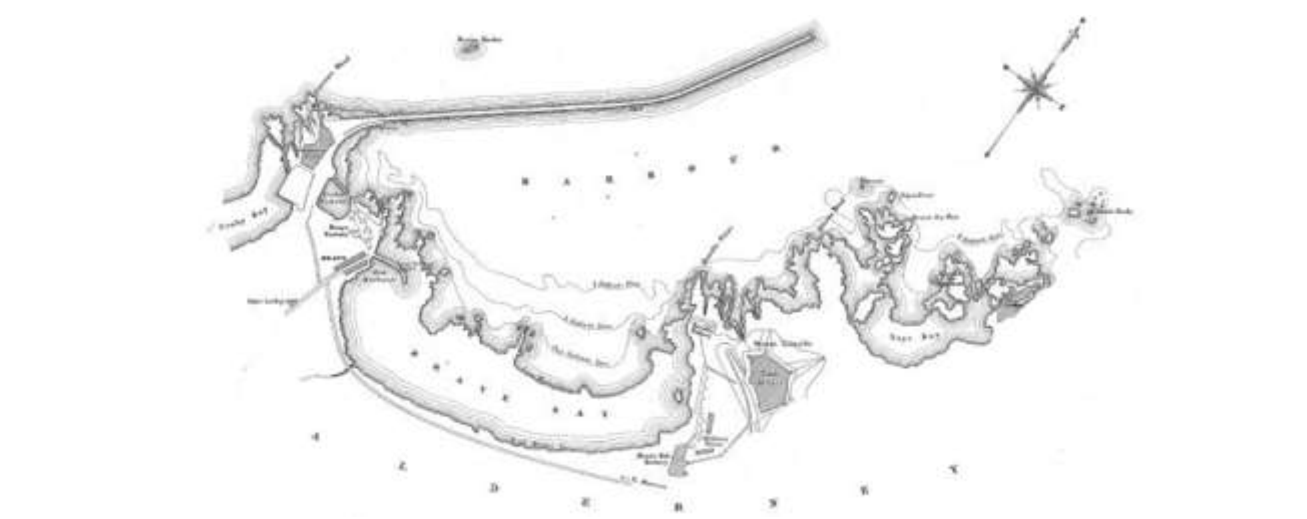


**Figure 6.7 Admiralty breakwater enclosing Braye Bay, Alderney**
*From Vernon-Harcourt (1873)*

Alderney island lies west of the harbour of Cherbourg in tidal streams where flows are compressed by the Cotentin peninsular giving: the Swinge to the west; and the Alderney Race to the east. The western coast of Alderney including Braye Bay is exposed directly to Atlantic waves.

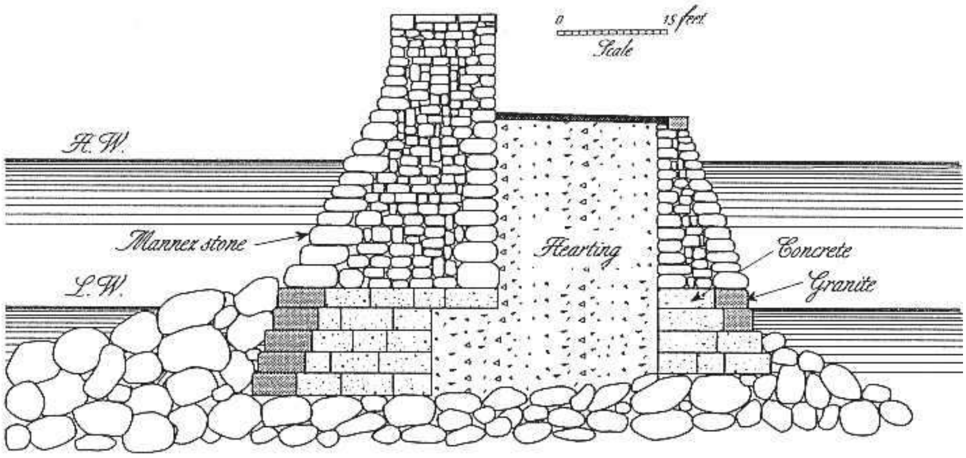



**Figure 6.8 Simplified section of Alderney breakwater wall**
*After Allsop & Shih (1990)*

Construction of the Admiralty breakwater in Braye Bay (Figure 6.7) started in 1847, to a design by James Walker, 2nd ICE President. The intended harbour enclosed Braye Bay by a breakwater of about 1450m length. The breakwater design included a mound to low water, surmounted by blockwork walls with rubble infill, Figure 6.8 and 6.9. Stone for mound and walls was quarried from the Mannez quarry on the opposite side of Alderney and transported by rail across the island. Granite cladding blocks were shipped in from Guernsey.

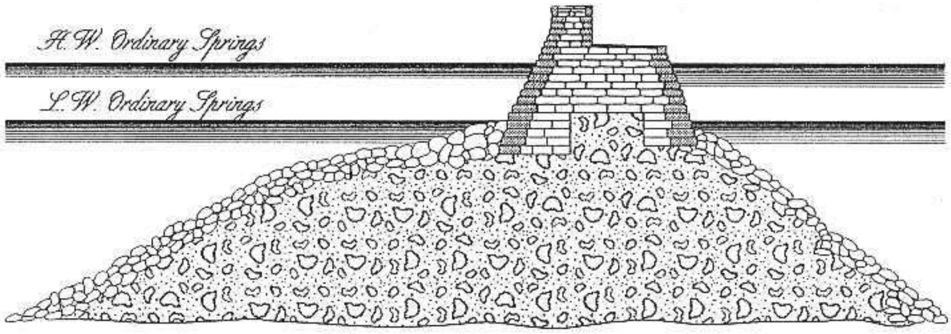



**Figure 6.9 Simplified section of outer length of Alderney breakwater mound and wall, showing extended mound**
*After Vernon-Harcourt (1885))*

By 1849, experience over two winters had shown up significant weakness in Walker's design with frequent breaches of the breakwater wall. The design section was amended from chainage 125m steepening the wall, blockwork was now set in Medina cement, and the foundation for the seaward face of the wall was started lower, Figure 6.10.

Moving out along the extending rubble mound, having used end-tipping hitherto, the new lower mound level, and wider footprint, required the use of hopper barges, in turn requiring a small construction harbour and a means of loading the barges. In the new works, the rubble mound was not disturbed below -3.7mLW (in the absence of the reflecting superstructure).

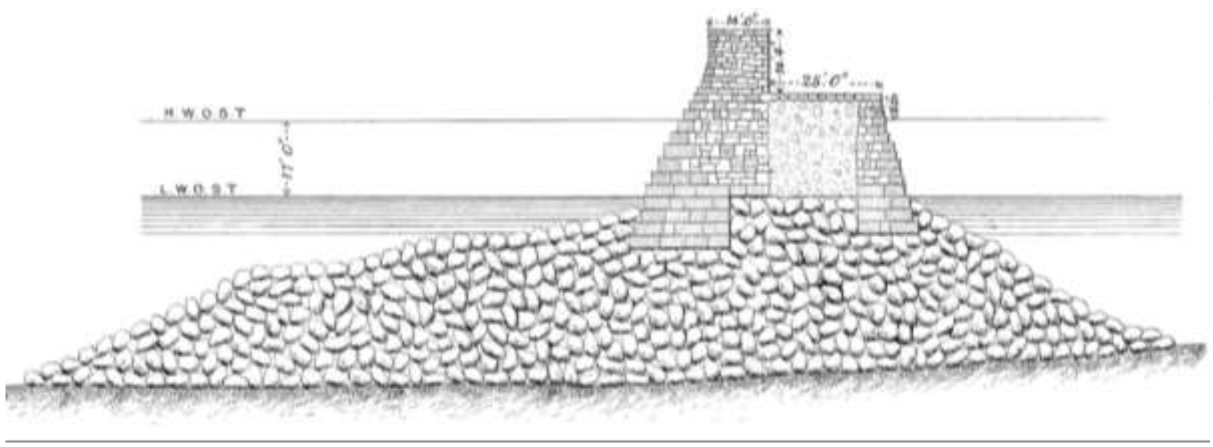



**Figure 6.10 Alderney breakwater at 300m from root**
*From Vernon-Harcourt (1885)*

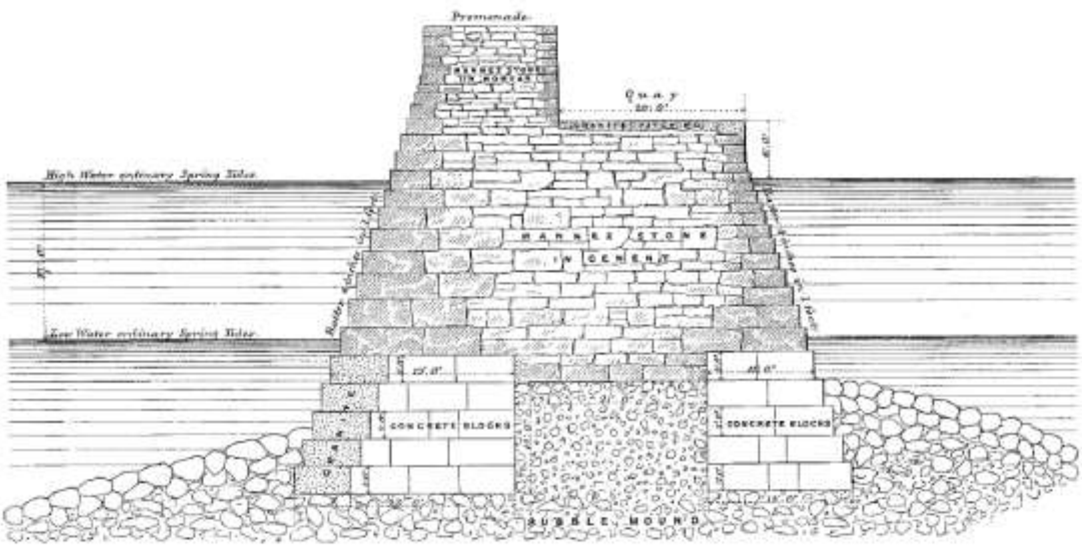



**Figure 6.11 Alderney breakwater superstructure at 830m from root**
*From: Vernon-Harcourt (1874)*

In 1852 (five years after construction started) following repeated breaches and cost increases, Sir Edward Belcher was sent by Admiralty to *"...go to Alderney harbour and report upon it*." Belcher concluded his discussion to Vernon-Harcourt (1873) 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.*"

This revised construction continued to 823m by 1856. The section design was then revised again, further lowering the wall foundation level, now easier with increased availability of divers, and cementing the wall fill, Figure 6.11.

Following (nominal) completion during 1864 to the full 1430m, repeated storms in 1865 to 1869 caused at least nine breaches through the superstructure. Despite repairs, by early 1870, there remained seven locations of damage. Vernon-Harcourt (1873) reported that Sir John Hawkshaw (President ICE) and Col. Sir Andrew Clarke were requested by Board of Trade *"to visit Alderney and to report on the best measures for securing permanently*", either the whole length (1430m) or an inner (870m) portion. Hawkshaw and Clarke noted instability of the mound and suggested removal of the promenade wall, and deposition of a large additional foreshore of rubble or concrete blocks. The government did not consider that the costs were merited, so no significant actions were taken.

About 300,000 tons of stone were tipped between 1864 and 1871, after which the Board of Trade decided to abandon the outer length. Partridge (2018) notes up to 20 breaches or defects by 1873. The main damage had been seaward of the 870m division. From 1873, repair and maintenance work covered only the inner length of 870m, and Partridge (2018) reports "*destruction of the seaward end*" by 1879 and "*outer section collapsed and submerged*" by 1889, leaving a submerged mound at about -4mLW.

**Figure 6.12 The Admiralty breakwater in 2014**
*Courtesy States of Guernsey*

For the shortened section, approximately 20,000 tons of stone were dumped annually, only formally ceasing in 1964, although it was interrupted during the German occupation, 1940-1945.

At many stages of tide, the mound induces wave breaking onto the wall. Direct wave impact on the wall shakes the breakwater, and cracks mortar joints. Impact pressures force water into the joints, and 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 gravel and sand on Little Crabby and Platte Saline beaches.

During winter 1989/90, storms battered the breakwater for six weeks. At its peak on 25/26 January 1990, the storm was about 1:25 years return, 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 pounding cracked the masonry facing, and a large cavity was formed in the wall which was breached by an explosive failure audible around Braye. An emergency procedure had previously been formulated, and repair work was underway within 10 days. The repair cost was estimated in 1990 at £1.1 million, equivalent to two years maintenance. Coode & Partners and HR Wallingford explored various potential solutions reported by Allsop *et al* (1991). Alternative approaches to protecting this breakwater were later described by Sayers *et al* (1998), and recently by Jensen *et al* (2017).

6 APPLICATION of ANALYSIS METHODS to SELECTED STRUCTURES

For the Safety Factor analysis here, four cross-sections have been defined running across the seabed and mound orthogonally to the wall, Figure 6.13. An additional section at chainage 670m is taken from HR Wallingford (1990), listed in Table 6.5. Depths and offsets from the wall toe are shown in Figure 6.14 for each section.

**Table 6.5 Section lines across Alderney breakwater used for Factors of Safety analysis**

| Section number | Chainage (m) from breakwater root at Fort Grosnez |
|---|---|
| A | 130 |
| B | 310 |
| C | 620 |
| HRW (1990) | 670 |
| D (breakwater head) | 870 |

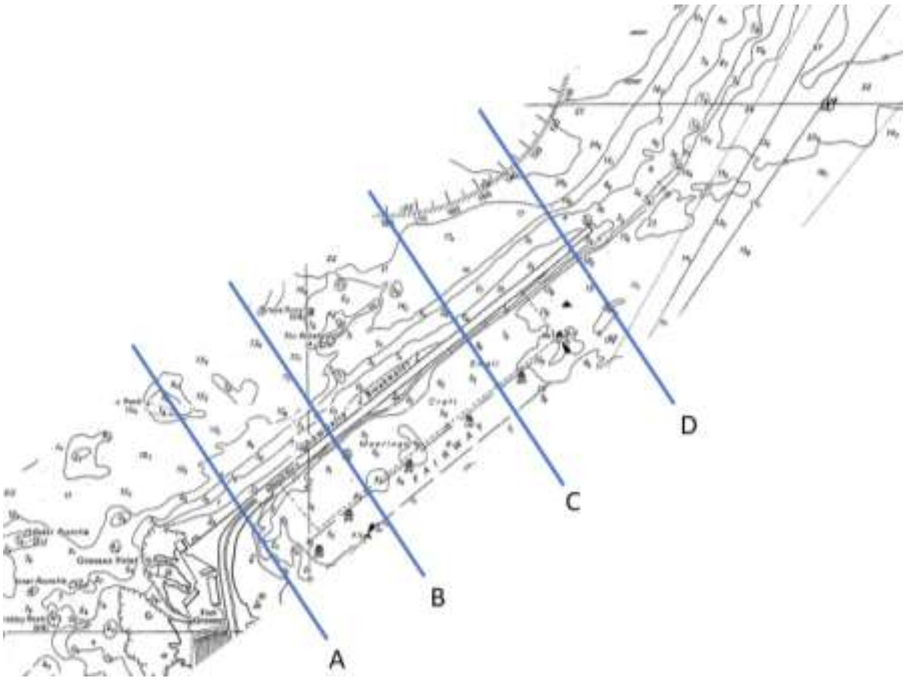



**Figure 6.13 Admiralty breakwater showing submerged mound and section lines**
*Chart courtesy States of Guernsey.*

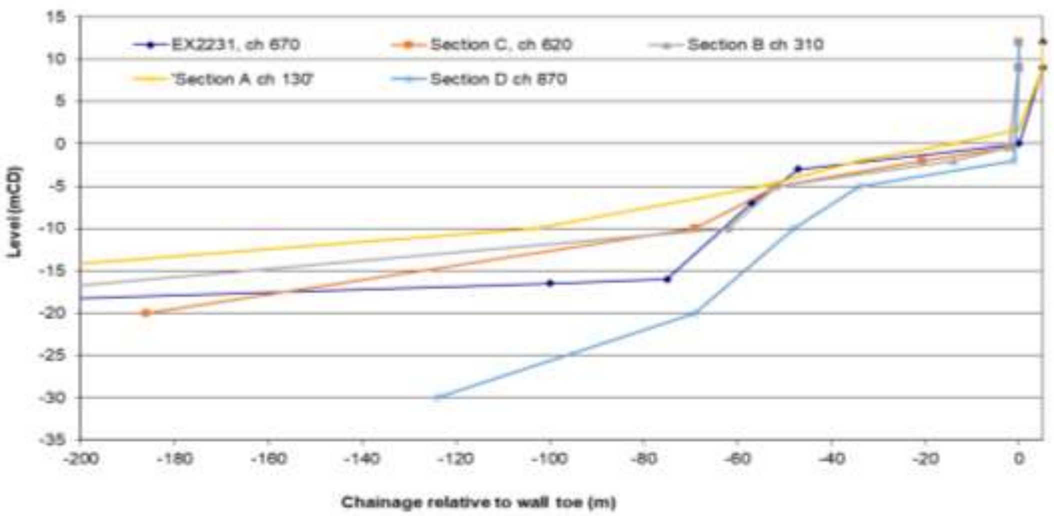



**Figure 6.14 Sections through Alderney breakwater**

### 6.3.2 Derivation of representative wave conditions

The extreme (astronomical) tidal range at Alderney is ~ 6.8m, from CD upwards. The main tidal levels listed on Admiralty chart 2845 are:

| | |
|---|---|
| Mean high water springs, MHWS | +6.3mCD |
| Mean high water neaps, MHWN | +4.7mCD |
| Mean low water neaps, MLWN | +2.6mCD |
| Mean low water springs, MLWS | +0.8mCD |

In their physical model study, HR Wallingford (1990) used three different test water levels: 6.1mCD; 3.15mCD; and 0.9mCD. Wave conditions at Alderney Breakwater are significantly influenced by the strong tidal flows in the approaches (the Swinge) which are at their greatest at around high and low water, tidal flows being lower and reversing at mid-tide. Wave heights at the breakwater are generally greatest at mid-tide, hence testing at +3.15mCD rather than at a higher water level. Wave conditions used for the testing at HR Wallingford in Table 6.6 were defined seaward of the approach bathymetry and mound. The wave transformation modelling took account of both local depths and effects of the high velocity currents. Wave periods are only given by HR Wallingford in EX2231 as mean period, $T_m$, so an additional column of peak periods, $T_p$, are given using a nominal conversion, $T_p/T_m \approx 1.1$.

**Table 6.6 Wave conditions at Alderney breakwater, from HR Wallingford (1990)**

| Return period: yrs | $H_s$ : m | $T_m$ : s | $T_p$ ; s |
|---|---|---|---|
| 1 | 6.0 | 11.0 | 12.1 |
| 5 | 7.0 | 11.4 | 12.5 |
| 20 | 7.4 | 11.9 | 13.1 |
| 50 | 8.4 | 12.7 | 14.0 |
| Extreme | 9.2 | 12.0 | 13.2 |

No return period was ascribed to the 'Extreme' condition at the time of the HR Wallingford study, but an approximate return period has been estimated here by fitting wave heights against return period for 1:1 to 1:50 year conditions on logarithmic axes in Figure 6.15. The resulting plot suggests that the 'Extreme' condition might represent a return period of approximately 1:150 year, so those conditions have been labelled as 1:150 year in the results tables that follow.

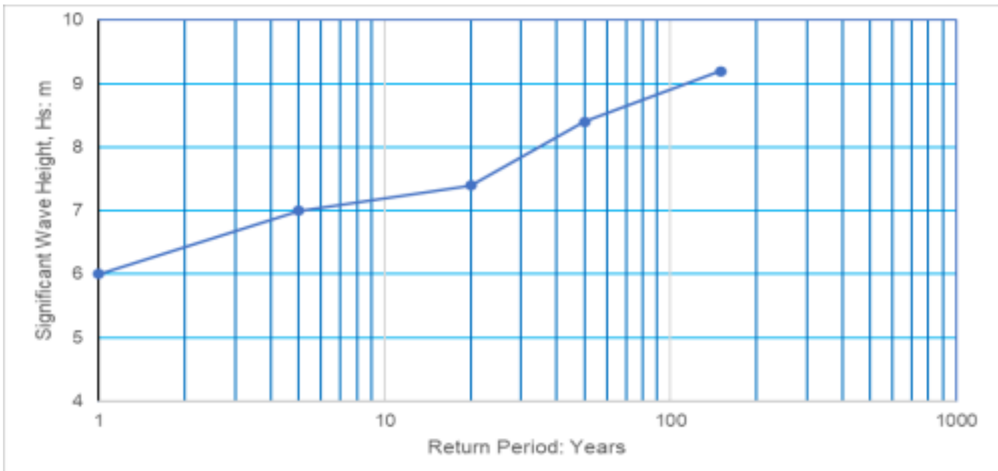



**Figure 6.15 Wave conditions from HR Wallingford tests**
*Data from HR Wallingford report EX2231*

To determine wave forces at the breakwater wall itself, incident waves must be transformed by shoaling and any depth-limited breaking approaching the breakwater. These calculations, and of breaking and wave loads, require various depths to be defined. One obvious 'depth' is at the toe of the breakwater wall at its intersection with the mound. Another 'depth' might be taken at the toe of the breakwater mound. The difficulty here is that the breakwater mound at Alderney was 'fed' with additional rock over approximately 90 years, so its toe has extended well beyond its original position in Figure 6.9, although it may also have eroded backwards over the last 50 or so years. Inshore of Braye Rocks, an average outer depth of -10mCD has been taken as representative, although this might be extended out to -20mCD or -30mCD along the outer part. In the sections in Figure 6.14, the breakwater 'mound' may be taken some 70m out from the wall to bed levels of around -10mCD down to -25mCD. Approach slopes are generally steep, particularly at about 50m out from the breakwater toe, so an indicative slope of 1:10 has been adopted for the following analysis.

The bed levels extracted above define bed slopes and levels 50m out at the 'Goda breaking position' (5 x $H_{so}$ seaward of the breakwater). The 'design' wave heights for this stage of analysis have been taken as the 1:20 year; 1:50 year; and 1:150 year conditions as in Table 6.7.

**Table 6.7 Results of wave transformation calculations, Alderney**

| Return period | Section | s.w.l. | $H_s$ | $T_p$ | Bed | Depth | $K_s$ | $H_{ss}$ | $H_{sb}$ | $H_{si}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| years | | mCD | m | s | mCD | m | | m | m | m |
| 20 | A | 3.5 | 7.4 | 13.1 | -8 | 11.5 | 1.02 | 7.5 | 10.4 | 7.5 |
| 20 | B / C | 3.5 | 7.4 | 13.1 | -12 | 15.5 | 0.97 | 7.2 | 11.9 | 7.4 |
| 20 | EX2231 | 3.5 | 7.4 | 13.1 | -18 | 21.5 | 0.94 | 6.9 | 13.6 | 7.4 |
| 20 | D | 3.5 | 7.4 | 13.1 | -25 | 28.5 | 0.92 | 6.8 | 17.2 | 7.4 |
| 50 | A | 3.5 | 8.4 | 14.0 | -8 | 11.5 | 1.04 | 8.7 | 11.1 | 8.7 |
| 50 | B / C | 3.5 | 8.4 | 14.0 | -12 | 15.5 | 0.99 | 8.3 | 12.8 | 8.4 |
| 50 | EX2231 | 3.5 | 8.4 | 14.0 | -18 | 21.5 | 0.95 | 8.0 | 14.6 | 8.4 |
| 50 | D | 3.5 | 8.4 | 14.0 | -25 | 28.5 | 0.92 | 7.8 | 17.2 | 8.4 |
| 150 | A | 3.5 | 9.2 | 13.2 | -8 | 11.5 | 1.02 | 9.4 | 10.5 | 9.4 |
| 150 | B / C | 3.5 | 9.2 | 13.2 | -12 | 15.5 | 0.97 | 9.0 | 12.0 | 9.2 |
| 150 | EX2231 | 3.5 | 9.2 | 13.2 | -18 | 21.5 | 0.94 | 8.6 | 13.8 | 9.2 |
| 150 | D | 3.5 | 9.2 | 13.2 | -25 | 28.5 | 0.92 | 8.4 | 17.1 | 9.2 |

*NB: As the 'input' waves approaching the breakwater had been derived by numerical modelling of wave refraction processes, those conditions will have included any shoaling, the results of $K_s$<1 have not been applied in the final calculation of $H_{si}$ in the last column.*

The waves in Table 6.7 have been shoaled from their offshore wave height using classical shoaling equations (checked against the SPM (1977) graphical method), and then checked for depth-limited breaking using Owen's (1980) equations summarised earlier in Figure 5.4. These calculations have been run for water levels of +6.1m and +3.5mCD, but only the latter are shown in Table 6.7 as the higher water level gave no significant increase in wave heights. Breaker limits over the approach slope ($H_{sb}$) are greater than incident waves, so little depth-limited breaking is expected before the mound.

### 6.3.3 Breakwater section

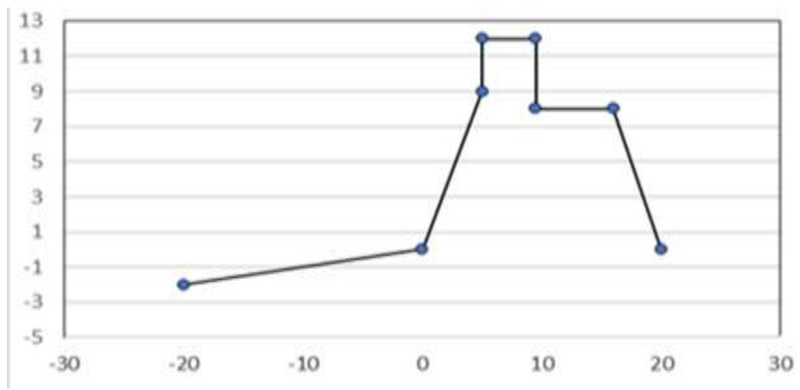



**Figure 6.16 Simplified cross-section of breakwater wall used in Factor of Safety analysis**

As previously discussed, the wall profile changed at various points during its construction. The wall face became steeper, and the wall toe and mound crest were lowered. In relation to wave height and wavelength, such changes are however relatively small, and will not materially alter wave effects on the wall, so the simplified section in Figure 6.16 will suffice for the analysis here. The parapet wall section is simplified to 4m x 4m, and the main wall body at 16m x 8m. The wall / mound interface is taken at 0mCD, and the wall crest at +12mCD. Taking blockwork at 5% porosity and stone of 2700kg/m$^3$ gives a net density of 2565kg/m$^3$.

The approach bed slopes are steep, although the seabed depths along most of the breakwater length are relatively large. Wave heights are however relatively large at $H_s* = H_{si}/h_s$= 0.4 to 0.8 for 1:50 year at all sections (except D where $H_s*$=0.3), giving 'large waves' in the PROVERBS classification. At $B* = B_{eq} / L_p \sim 0.3$ to 0.5, the foundation mound is classed as 'moderate to wide mound'.

**Figure 6.17 Waves breaking impulsively along mid-section**
*Model tests at HR Wallingford ~1990*

These calculations suggest that most severe breaking occurs over the mid-length, being broken at Section A (ch. 130m) by the shallow depths near the root, see example photo from model tests in Figure 6.17. Contrarily, depths at the head (Section D, ch. 870) are greater, and the mound is lower, so impulsive breaking over the mound is less prevalent.

In these calculations, the foreshortened bathymetry in the HR Wallingford (1990) tests artificially increased $H_{bc}$, apparently reducing values of $P_i$ and $P_b$. These distorted values are therefore not shown in Table 6.8.

Changing the water level can however have a significant effect on the breaking results, so the calculations below include results for water levels of +3.5mCD and +6.1mCD.

**Table 6.8 Wave breaking, Alderney**

| Return period | Section | $H_s$ | $T_p$ | Bed | Depth | $H_{bc}$ | $H_{si}/H_{bc}$ | $P_i$ | $P_b$ |
|---|---|---|---|---|---|---|---|---|---|
| years | | m | s | mCD | m | m | | % | % |
| At water level = +3.5mCD | | | | | | | | | |
| 20 | A | 7.4 | 13.1 | -8 | 11.5 | 14.0 | 0.56 | 0.2 | 0.0 |
| 20 | B / C | 7.4 | 13.1 | -12 | 15.5 | 6.3 | 1.17 | 23.3 | 9.1 |
| 20 | D | 7.4 | 13.1 | -25 | 28.5 | 6.6 | 1.12 | 20.3 | 7.9 |
| 50 | A | 8.4 | 14.0 | -8 | 11.5 | 15.0 | 0.60 | 0.4 | 0.1 |
| 50 | B / C | 8.4 | 14.0 | -12 | 15.5 | 6.4 | 1.32 | 31.7 | 12.6 |
| 50 | D | 8.4 | 14.0 | -25 | 28.5 | 6.6 | 1.26 | 28.7 | 11.3 |
| 150 | A | 9.2 | 13.2 | -8 | 11.5 | 14.1 | 0.69 | 1.4 | 0.5 |
| 150 | B / C | 9.2 | 13.2 | -12 | 15.5 | 6.3 | 1.45 | 38.8 | 15.6 |
| 150 | D | 9.2 | 13.2 | -25 | 28.5 | 6.6 | 1.39 | 35.6 | 14.2 |
| At water level = +6.1mCD | | | | | | | | | |
| 20 | A | 7.4 | 13.1 | -8 | 14.1 | 4.6 | 1.63 | 47.1 | 19.1 |
| 20 | B / C | 7.4 | 13.1 | -12 | 18.1 | 4.2 | 1.76 | 52.4 | 21.3 |
| 20 | D | 7.4 | 13.1 | -25 | 31.1 | 7.9 | 0.93 | 10.1 | 3.8 |
| 50 | A | 8.4 | 14.0 | -8 | 14.1 | 4.6 | 1.88 | 56.9 | 23.2 |
| 50 | B / C | 8.4 | 14.0 | -12 | 18.1 | 4.2 | 1.99 | 60.4 | 24.7 |
| 50 | D | 8.4 | 14.0 | -25 | 31.1 | 8.0 | 1.05 | 16.5 | 6.3 |
| 150 | A | 9.2 | 13.2 | -8 | 14.1 | 4.6 | 2.04 | 61.9 | 25.4 |
| 150 | B / C | 9.2 | 13.2 | -12 | 18.1 | 4.2 | 2.19 | 65.8 | 27.1 |
| 150 | D | 9.2 | 13.2 | -25 | 31.1 | 7.9 | 1.16 | 22.7 | 8.8 |

### 6.3.4 Calculations of loads

As before, horizontal forces have been calculated using methods by Ito and Goda using the simple empirical methods outlined in Chapter 5.

The total horizontal forces, $F_h$, in Table 6.9 are calculated by integrating pressures $p_1$, $p_2$, and $p_3$ over the height $h_f$ of the front face. Similarly, the up-lift force is calculated by integrating from $p = p_u$ at the front edge to $p = 0$ at the rearward edge, giving $F_u = 0.5\ p_u\ B_c$. All forces (and pressures) are quoted as 1/250 values, $F_{1/250}$, equal to the average of the largest 4 in 1000 events).

As is often seen, horizontal forces calculated by Ito's method are larger than those by Goda's method, reflecting the inherent conservatism of the earlier and more simplistic approach.

It is interesting to note that these wave loads are generally higher for the lower water level used here (+3.5mCD) than for the higher water level (+6.1mCD), matching the local experience that the more severe conditions at Alderney breakwater occur around the mid-water levels, not at HW.

**Table 6.9 Wave loads by Ito (1971) and Goda (1985, 2000)**

| Return period | Section | $p_{av}$ | $F_{h(Ito)}$ | $\eta^*$ | $p_1$ | $p_2$ | $p_3$ | $F_{h(Goda)}$ | $F_{u(Goda)}$ |
|---|---|---|---|---|---|---|---|---|---|
| years | | kN/m$^2$ | kN/m | m | kN/m$^2$ | kN/m$^2$ | kN/m$^2$ | kN/m | kN/m |
| At water level = +3.5mCD | | | | | | | | | |
| 20 | A | 113 | 2263 | 21.1 | 163 | 157 | 162 | 1950 | 710 |
| 20 | B / C | 81 | 1953 | 20.0 | 148 | 138 | 145 | 1760 | 670 |
| 20 | D | 53 | 1968 | 20.0 | 130 | 106 | 125 | 1530 | 670 |
| 50 | A | 147 | 2939 | 24.3 | 186 | 181 | 186 | 2230 | 815 |
| 50 | B / C | 102 | 2444 | 22.7 | 173 | 165 | 171 | 2060 | 760 |
| 50 | D | 66 | 2425 | 22.7 | 152 | 129 | 148 | 1800 | 760 |
| 150 | A | 169 | 3374 | 26.2 | 199 | 192 | 198 | 2380 | 880 |
| 150 | B / C | 120 | 2876 | 24.8 | 192 | 181 | 189 | 2290 | 830 |
| 150 | D | 76 | 2823 | 24.8 | 165 | 135 | 159 | 1945 | 830 |
| At water level = +6.1mCD | | | | | | | | | |
| 20 | A | 90 | 1792 | 20.3 | 152 | 144 | 150 | 1810 | 680 |
| 20 | B / C | 72 | 1740 | 20.0 | 141 | 130 | 137 | 1670 | 670 |
| 20 | D | 50 | 1864 | 20.0 | 127 | 100 | 120 | 1490 | 670 |
| 50 | A | 117 | 2338 | 23.5 | 184 | 176 | 181 | 2190 | 790 |
| 50 | B / C | 90 | 2170 | 22.7 | 165 | 154 | 161 | 1950 | 760 |
| 50 | D | 62 | 2291 | 22.7 | 149 | 123 | 142 | 1740 | 760 |
| 150 | A | 134 | 2689 | 25.4 | 201 | 191 | 198 | 2400 | 850 |
| 150 | B / C | 106 | 2547 | 24.8 | 181 | 167 | 176 | 2140 | 830 |
| 150 | D | 72 | 2662 | 24.8 | 161 | 127 | 152 | 1880 | 830 |

### 6.3.5 Calculations of Factors of Safety

The simplest stability analysis compares horizontal loads (and up-lift) versus sliding resistance given by weight and friction. This requires making some simplifying assumptions, but these are generally robust and easily interpreted. The first stage in estimating sliding resistance is to compute a representative section with appropriate weights for the breakwater wall. For Alderney, the wall section can be simplified into three parts, see Figure 6.16:

- The rubble mound, approximately 120m wide at the base and from about -12mCD or -25mCD up to 0mCD, so about 12 to 25m high.
- The main wall section founded at about 0mCD and rising to the walkway at +8mCD, so about 8m high. At mid-height, the wall is about 16m wide (front to back), so occupies about 128m$^2$.
- The main wall is topped by a parapet wall about 4m high, and 4m wide, so occupying about 16m$^2$.

Most of the outer section is formed by close-fitting blocks, so the weight of the main wall and parapet is 144m$^3$/m x 2.565t/m$^3$ = 370tonne/m. Buoyant up-lift will act to either 6.1mCD or 3.5mCD. Converting weight and buoyant up-lift to kN/m, we have net weight forces of 2700kN/m and 3060kN/m at water levels of 6.1mCD and 3.5mCD respectively

For sections B and C (chainages 310m and 620m), assuming a friction coefficient μ = 0.78 see Hutchinson *et al* (2010), gives net sliding resistances of 2110 kN/m and 2390 kN/m respectively to set against the wave loads in Table 6.10. The resulting Factors of Safety, without and with the effect of wave uplift, are summarised in Tables 6.10 and 6.11. The coefficient of friction μ = 0.78 was derived from full-scale pull tests on a roughened caisson base onto rock (Hutchinson *et al,* 2010).

**Table 6.10 Factor of Safety (sliding) for sections B /C, no wave uplift**

| Return period (years) and section | Static water level (mCD) | Sliding resistance (kN/m) | Wave force, $F_h$ (kN/m) | Factor of Safety (-) |
|---|---|---|---|---|
| 20 B/C | 3.5 | 2280 | 1760 | 1.29 |
| 50 B/C | 3.5 | 2280 | 2060 | 1.10 |
| 150 B/C | 3.5 | 2280 | 2290 | 0.99 |
| 20 B/C | 6.1 | 1870 | 1670 | 1.12 |
| 50 B/C | 6.1 | 1870 | 1950 | 0.96 |
| 150 B/C | 6.1 | 1870 | 2145 | 0.87 |

Taking the more optimistic scenario, without wave-driven up-lift forces as in Table 6.10, and assuming the breakwater wall to act monolithically, then the wall should slide under a 1:50 year wave condition at a high water level of +6.1mCD, but should just resist sliding at the lower water level of +3.5mCD. Factors of Safety calculated in Table 6.11 are lower, but those calculations have not included the restraining weight of additional material behind the simple section considered here, i.e. the work area at the breakwater root and the inclined slipway, all of which will substantially increase sliding resistance.

**Table 6.11 Factor of Safety (sliding) for sections B /C, with wave uplift**

| Return period (years) and section | Static water level (mCD) | Sliding resistance (kN/m) | Wave force, $F_h$ (kN/m) | Factor of Safety (-) |
|---|---|---|---|---|
| 20 B/C | 3.5 | 1755 | 1760 | 1.0 |
| 50 B/C | 3.5 | 1680 | 2060 | 0.82 |
| 150 B/C | 3.5 | 1630 | 2290 | 0.71 |
| 20 B/C | 6.1 | 1350 | 1670 | 0.81 |
| 50 B/C | 6.1 | 1275 | 1950 | 0.65 |
| 150 B/C | 6.1 | 1218 | 2145 | 0.57 |

If Goda up-lift forces are included (as in Table 6.11), then the idealised structure will fail by sliding under all of these conditions. It might be argued that it is however unlikely that the up-lift pressures will act on the 'base' of the breakwater superstructure, indeed that such upward pressures will at least partially dissipate within the rubble fill.

**Table 6.12 Factor of Safety (overturning) for sections B / C, no wave uplift**

| Return period (yrs) and section | Static water level (mCD) | Overturning resistance, kNm/m | Overturning moment, $M_{total}$, kNm/m | Factor of Safety |
|---|---|---|---|---|
| 20 B/C | 3.5 | 39500 | 7920 | 5.0 |
| 50 B/C | 3.5 | 39500 | 8080 | 4.9 |
| 150 B/C | 3.5 | 39500 | 8200 | 4.8 |
| 20 B/C | 6.1 | 39500 | 13100 | 3.0 |
| 50 B/C | 6.1 | 39500 | 13300 | 3.0 |
| 150 B/C | 6.1 | 39500 | 13400 | 2.9 |

Stability of the nominal section against overturning around the rear heel of the wall is assessed here by apportioning 'Goda pressures' over the front face, and including effects of buoyancy. As for sliding, the analysis has been repeated with and without wave uplift forces.

**Table 6.13 Factor of Safety (overturning) for sections B / C, with wave uplift**

| Return period (yrs) and section | Static water level (mCD) | Overturning resistance, kNm/m | Overturning moment, $M_{total}$, kNm/m | Factor of Safety |
|---|---|---|---|---|
| 20 B/C | 3.5 | 39500 | 17000 | 2.3 |
| 50 B/C | 3.5 | 39500 | 18200 | 2.2 |
| 150 B/C | 3.5 | 39500 | 19300 | 2.0 |
| 20 B/C | 6.1 | 39500 | 22000 | 1.8 |
| 50 B/C | 6.1 | 39500 | 23400 | 1.7 |
| 150 B/C | 6.1 | 39500 | 24500 | 1.6 |

The overturning results in Tables 6.12 and 6.13 are reassuring for overall stability with overturning FoS always above 1.6 and generally above 3.0 for the no-uplift cases. This stability is assisted by the relatively large width of the superstructure increasing the resistance moment; and the moderate crest level, keeping the centre of wave forces low.

### 6.3.6 Impulsive loads

Impulsive loads have been calculated in Table 6.14 using Cuomo *et al*'s (2010, 2011) methods from which it can be seen that impulsive loads greater than (say) 2x the Goda load will only act for a duration of $\approx 0.01.T_m$, so of order 0.1s. This is however probably of sufficient duration to cause motion of unbonded blocks, but is unlikely to cause noticeable motion to the wall section as a whole.

**Table 6.14 Impulsive and quasi-static force maxima, methods of Cuomo et al (2010**

| Return period (yrs) and section | Static water level (mCD) | Impulsive force, $F_{imp}$, kN/m | Quasi-static force, $F_{qs}$, kN/m | $F_{imp}$ / $F_{qs}$ |
|---|---|---|---|---|
| 20 A | 6.1 | 8670 | 2720 | 3 |
| 20 B/C | 6.1 | 3170 | 2640 | 1.2 |
| 20 D | 6.1 | 730 | 2640 | - |
| 50 A | 6.1 | 14420 | 3650 | 4 |
| 50 B/C | 6.1 | 5990 | 3410 | 1.8 |
| 50 D | 6.1 | 7850 | 3410 | - |
| 150 A | 6.1 | 17220 | 4260 | 4 |
| 150 B / C | 6.1 | 8160 | 4080 | 2 |
| 150 D | 6.1 | 10340 | 4080 | 1.2 |
| | | | | |
| 20 A | 3.5 | 33850 | 2940 | >10 |
| 20 B/C | 3.5 | 10800 | 2640 | 4 |
| 20 D | 3.5 | 6300 | 2640 | 2.4 |
| 50 A | 3.5 | 50500 | 3910 | >10 |
| 50 B/C | 3.5 | 16500 | 3410 | 5 |
| 50 D | 3.5 | 10700 | 3410 | 3 |
| 150 A | 3.5 | 57800 | 4540 | >10 |
| 150 B / C | 3.5 | 20200 | 4080 | 5 |
| 150 D | 3.5 | 13600 | 4080 | 3 |

A further aspect of impulsive loads is that they are of limited spatial extent, so even if effective in causing (short duration) excess loadings, these will be confined to limited areas. That said, the low Factors of Safety, do provide justification for the historical failures, and the high predictions of impulsive loads in Table 6.14 indicate the significant loadings to which this breakwater is potentially subject.

### 6.3.7 Conclusions

Over the years from 1850 onwards, Alderney breakwater has suffered many local failures. These might often be precipitated by removal of individual blocks from low down in the wall, and/or by loss of foundation support to uncemented (or poorly cemented) blockwork.

The simplified empirical tools used in the analysis here, particularly the simplistic representation of structural resistance, cannot therefore reliably predict such complex failures, but the results of the calculations run here do inform us on the general failure likelihood.

Loads have been calculated in this analysis for return periods of 1:20, 1:50 and 1:150 years. Given the breakwater life to 2020, about 160 years, the probabilities of matching or encountering conditions of these severities will be above 65% for the extreme case, and 96% for the 1:50 year condition. It would be reasonable to describe the encounter of this latter event over the breakwater life as ‘certain’ and of the ‘1:150 year’ event as ‘very likely’.

The simplest analysis of (quasi-static) sliding gives Factors of Safety close to (but below) unity for both of these wave conditions. Whilst the reliability of the wave load calculations will be good, it is likely that the stability analysis used here will have omitted or under-estimated various restraining mechanisms, particularly load-sharing along the wall under impulsive loads. Those loads will however have certainly caused distress to the wall, especially where variations in local support or bonding may lead to potential local failures. Given the high probabilities of such loads, it is perhaps surprising that failures have not continued in recent years, an indication that the continuing repair / rehabilitation measures over the years have steadily increased the resistance of the wall.

## 6.4 Case study 3, Dover

### 6.4.1 History of Dover (outer) harbour breakwaters

A National Harbour had been mooted at Dover for many years. Wilson (1919) noted that the 1840 Royal Commission favoured a deep-water harbour in Dover Bay of 450 acres (1.82km$^2$), estimated cost of £2,000,000. The 1844 Royal Commission re-considered whether to establish a harbour of refuge here. Whilst the harbour at Dover was in theory for refuge of civilian vessels, military purposes were clear from the start, and the Admiralty retained seminal influence throughout its development. This second Commission accepted the proposed site and layout of the new outer harbour. A third Commission in 1845 considered plans by eight leading engineers for a harbour of some 520 acres (2.1km$^2$) out to 7 fathoms (12.8m).

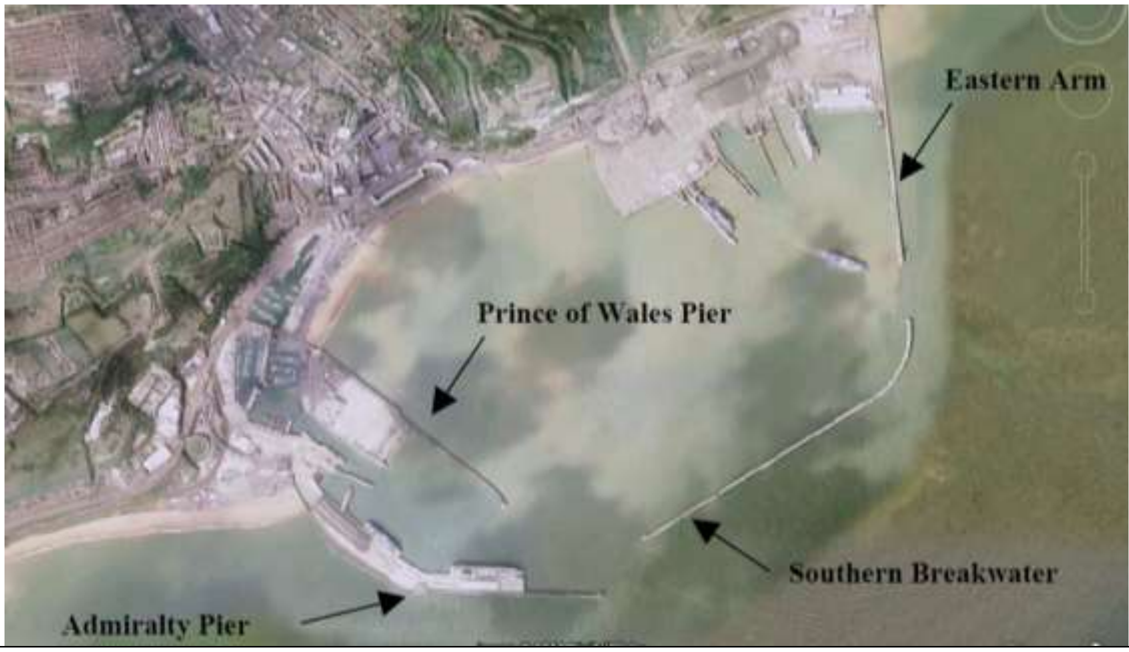



**Figure 6.18 Dover Harbour, present day**
*Courtesy HR Wallingford and Google Maps*

The Commissioners reported in 1846 in favour of Rendel’s design with a vertical wall. Vernon-Harcourt (1885) noted damage to sloping breakwaters at Cherbourg and Plymouth, but also the paucity of experience in concrete. But given the chalk bottom, absence of local rock, “*and a moderate depth, the upright wall was the best system to adopt*”. A series of contracts in 1847, 1854 and 1857 extended Admiralty Pier to 640m by 1871. Admiralty Pier was formed by 7-8 ton concrete blocks with stone facings on the outer faces. Wilson (1919) noted that about 300m of the main parapet wall (a single column of blocks, about 1.5m thick, with a slight recurve) was swept away in January 1877. The slender nature of the up-stand wall, and absence of tensile reinforcement against bending must surely have contributed. The damaged section was rebuilt with a significantly thicker (about 3.3m) vertical face.

This single pier did not however give adequate shelter from easterlies, and in 1895, Coode, Son & Matthews were requested to prepare surveys and drawings to facilitate expansion to the full Admiralty Harbour (Figure 6.18) by:

- Extension of Admiralty Pier by a further 610m;
- A detached breakwater, the South Breakwater, of 1284m;
- The Eastern Arm of 1012m.

This revised layout altered the length and overlap of the Admiralty Pier extension, and the position and width of the Eastern entrance to improve access and reducing siltation. The Coode design was rapidly approved by the Admiralty, and a construction contract was let to S Pearson & Son in November 1897.

The Eastern Breakwater, termed the East Arm, projects south for about 1km in section similar to the Admiralty Pier Extension, although the parapet wall was lower at +8.8m LWOST). Foundation blocks for the East Arm wall were laid direct on the chalk inshore, or the chalk marl / flint matrix further seaward, down to -16.2m LWOST. The East Arm was intended to provide berthing, so the harbour face was vertical with timber fenders, and an L-shaped head provided wave shelter along the inner face.

The South Breakwater, also termed the Island Breakwater, runs 1284m parallel to the shoreline. Placement of blocks started short of the eastern end, allowing adjustment of the eastern entrance based on wave and flow experience during construction. A curved length connected the eastern end to the main run using curved (radial) blocks to preserve block tightness.

The new walls were formed by 24-40t concrete blocks (2.3m wide and 1.8m high, depth from 2.4 to 4m) to accommodate the 12:1 batter and ensure adequate bonding, Figure 6.19. Jointing was strengthened by half-height joggle joints, filled by 4:1 concrete rammed into canvas bags. Around the outer ends, tensile connection was provided by bull-headed rails turned down at the ends and let into chased channels / holes filled with 2:1 cement mortar.

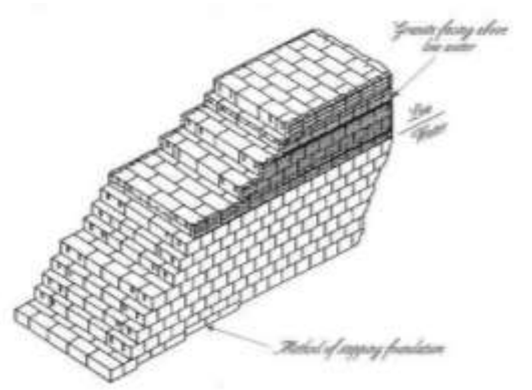


**Figure 6.19 Schematic of Dover breakwater blockwork**
*After Allsop (2009) after Wilson (1919)*

For the foundation layers, underwater blocks were set by divers, placed tightly without mortar. The Eastern Pier and Admiralty Pier Extension carried parapet walls, but overtopping protection was not needed on the South Breakwater as mooring against its inside face was not envisaged.

Ahead of block placing, the seabed was prepared by excavating 1.5m of surface material. The chalk or chalk / flint matrix was loosened by a cast-iron breaker dropped from the leading Goliath. The final 0.3m was removed using a diving bell excavating a strip for two rows of blocks. The bell passed over each strip firstly to give a coarse levelling, then a second pass for final levelling. Block-setting was supervised by two helmet-divers, blocks being placed hard against their neighbours to ensure an even base for subsequent blocks. Bag joggles were placed by divers or from within the bell. Blocks above were set by masons during the 2-3 hours of low water on spring tides. All upper courses were set / bedded in 2:1 Portland cement mortar so all lower joints were caulked by sacking or rope, pointed in neat (quick-setting) cement, to avoid loss of jointing or bedding mortar downward.

### 6.4.2 Derivation of representative wave conditions

For a residual life assessment study for Dover Harbour Board (DHB), HR Wallingford (2011) analysed wave loads on different parts of the main breakwaters at Dover. Selected calculations are repeated and extended here from calculations by the author within that study.

**Table 6.15 Wave conditions at Dover, after HR Wallingford (2011)**

| Return period (year) | Wave height, $H_s$ (m) | Wave period, $T_m$ (s) | Direction (°N) |
|---|---|---|---|
| 1 | 4.3 | 7.3 | 210 |
| 10 | 5.2 | 8.2 | 215 |
| 50 | 5.8 | 8.6 | 215 |
| 100 | 6.0 | 8.8 | 215 |
| 200* | 6.2 | 10.0 | - |
| 500* | 6.5 | 10.3 | - |

** These results were extracted by the author from his original calculations, prepared for but not shown in HR Wallingford (2011)*

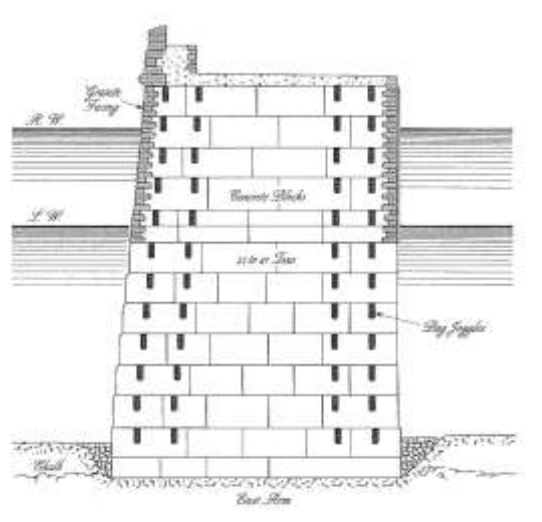


**Figure 6.20 Dover breakwater section**
*HR Wallingford (2011) after Wilson (1919)*

The range of water levels in the 2011 study covered return periods of 1-1000 years at dates of 2000 and 2060, giving water levels of: 7.4mCD up to 9.6mCD (0mODN = 3.67mCD). Wave conditions were extracted from previous wave modelling to give predicted wave heights ($H_s$) period ($T_m$) and direction (°N) for return periods from 0.1 year to 100 year. As might be expected from the exposure, the largest waves are from the South and South-West, and are likely to hit the Admiralty pier extension at normal incidence, $\beta \approx 0°$, summarised in Table 6.15.

### 6.4.3 Breakwater section

The Dover section analysed is far simpler than those for Wick or Alderney with no rubble mound or reef, Figure 6.20. Even so, some slight simplifications were made for this analysis. A seabed level at -11mCD was chosen, with a wall crest at +15mCD. The wall was taken as vertical on both faces and of width 15m. The slight batter will neither alter the loading materially nor the sliding resistance.

The HR Wallingford (2011) analysis had used a precautionarily light density of $\rho_c$= 2.14t/m$^3$ for the assemblage of concrete blocks, giving some degree of pessimism to the stability calculations. The section weight (dry) was 835 t/m. The effects on stability of a range of water levels were explored up to +9mCD.

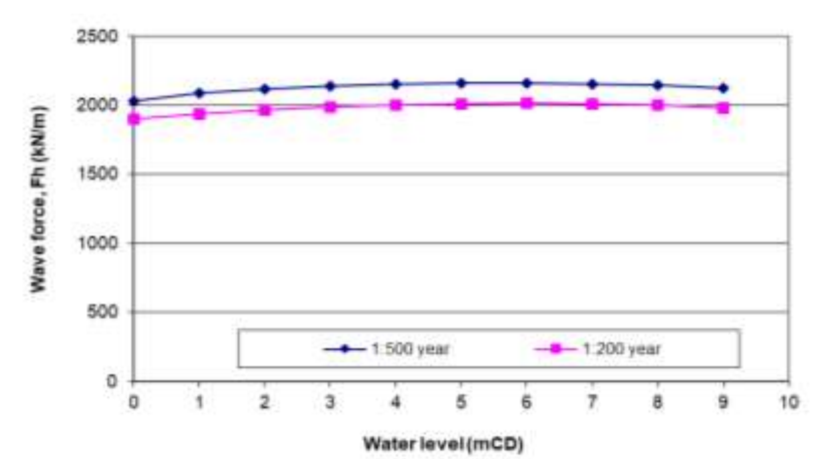



**Figure 6.21 Total horizontal wave force, $F_h$**

### 6.4.4 Calculation of wave loads

Wave loads were calculated using Goda's (1985, 2000) method, primarily to give the total horizontal (sliding) force. The seabed here is relatively flat, and this breakwater includes no berm or mound, so no impulsive loads are expected.

For the 1:200 and 1:500 year returns, horizontal loads alter very slightly with water level, Figure 6.21, reaching a maximum at a frequent water level of +6mCD.

### 6.4.5 Calculation of Factors of Safety

The final stage in the initial Factor of Safety analysis was to calculate *FoS* against sliding using a friction coefficient of μ=0.8. The (slight) increase over the previous value (μ=0.78) reflects the additional restraint where joggle bags cross layers, and any steps between layers of blocks. In all instances analysed, the Factors of Safety shown in Figure 6.22 remain well above unity. For $\mu = 0.8$, *FoS*> 1.5 for all cases modelled.

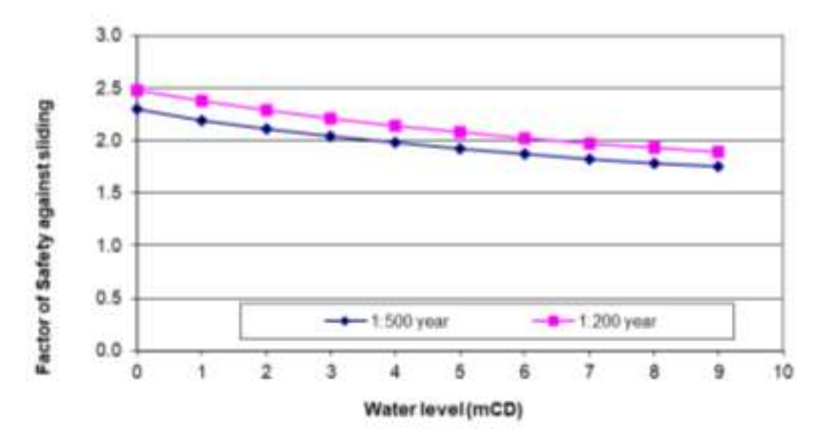



**Figure 6.22 Factors of Safety against sliding, μ = 0.8**

This analysis (and that on overturning discussed below) are surprisingly reassuring, given that even Sainflou's (1928) wave load analysis was only available in 1928 and Goda's first version was only first published in English in 1974 (Goda, 1974). The original designers probably therefore had no quantitative load prediction method other than their previous experience!

### 6.4.6 Further calculations

After the stability calculations reported previously by Allsop & Bruce (2020b), the stability analysis was expanded to include overturning moments, and to calculate possible wave loads that destroyed the (rather slender) Admiralty Pier parapet wall in January 1877.

**Overturning**

The sliding analysis above was extended to include overturning moments around the rear heel, and thus to calculate factors of safety on overturning. Results of *FoS* are plotted in the same format in Figure 6.23 with *FoS* > 6 for all conditions considered, demonstrating the relatively mild exposure, and the advantage of the relatively wide wall ($B_c \approx 15$m).

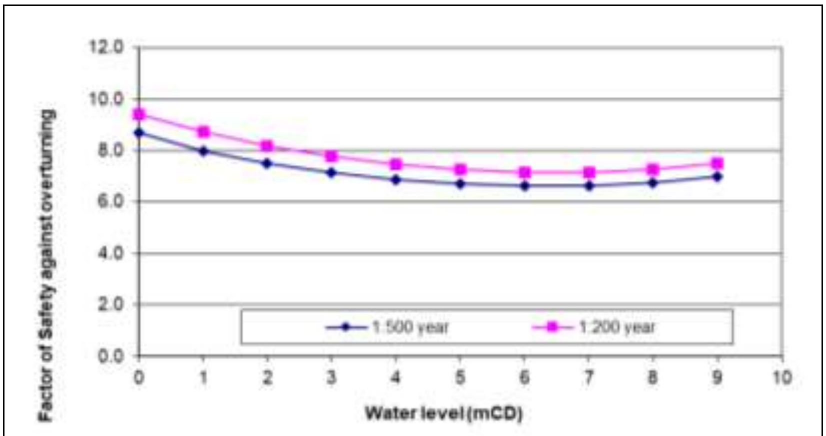



**Figure 6.23 Factors of Safety against overturning**

**Admiralty Pier parapet wall**

In contrast to the high levels of stability for the new construction, Wilson (1919) described a previous failure of 300m of parapet wall on Admiralty Pier under "*an exceptionally heavy south-west gale on 1 January 1877*". This upstand was 1.5m thick, formed by a single column of blocks. The effects of wave forces were probably increased by a slight recurve which will have in turn increased the overall wave force by a further change of wave momentum, and will have applied an up-lift component, this reducing the net resistance. As no details of either wave conditions, nor of the recurve geometry, are available, the analysis here is relatively simplistic.

In calculating the (Goda) wave pressures on the wall in 6.4.4 above, the average wave pressures on the upper wall were of order $p_{av} \approx 70$kPa. Applying that over a wall height of (say) 4m gives a sliding force of $F_h \approx 280$ kN/m. In contrast a wall 1.5m thick of blocks of $\rho_c = 2.2$t/m$^3$ would weigh 13.2t/m, giving a sliding resistance with $\mu = 0.8$ of about 105 kN/m, so a factor of safety of $FoS \approx 0.37$, well below unity even without factoring in any up-lift force, a fairly conclusive demonstration of the high probability of failure.

The parapet wall was rebuilt to a width of 3.3m, which would have given $FoS \approx 0.83$ again using sliding resistance of $\mu = 0.8$. This is clearly an improvement, but probably not enough to resist another direct hit unless the wall was structurally connected along its length, and into the rest of the crown wall. This length was later re-engineered during the major expansion by Pearson after 1892.

## 6.5 Conclusions from the Case Studies

The early failure of the end of Wick breakwater was due primarily to the large waves at Wick, compounded by the steep bed slopes over Crane Rocks and mound that shoaled those waves onto the breakwater, and delayed depth-limited breaking. Quasi-static wave loads onto the wall were therefore large enough to slide and/or overturn the solid monolith at the breakwater end, even without any impulsive loads (which would have exceeded the quasi-static loads already able to fail the breakwater).

At Alderney, waves are lower than at Wick, but wave periods are longer and the rubble mound is high relative to frequent water levels so that waves break severely onto the breakwater wall. These loads may be enough to move the wall, particularly when including some contribution from wave uplift and/or impulsive loads. The historic weaknesses and local damage to Alderney Breakwater will have been due to the very high impulsive loads, movement of the rubble mound, and loss of stability (bonding) of the lower blockwork. In early failures this was probably compounded by loss of fill from below the (uncemented) seaward wall.

At Dover, waves are smaller than the other two sites considered here, but most importantly the designer avoided any mound. Founding this vertical wall at seabed level removed any occurrence of the impulsive wave loadings so troublesome at Wick and Alderney.

This study has applied empirical analysis techniques developed over the last 25 years to breakwaters built between 1850 to 1910 to analyse the stability of three structures. This has required some simplifications, but even so, the Factor of Safety analysis has successfully matched their performance.

A simple analysis of forces and stability for the parapet wall on Admiralty Pier that failed in 1877 has been shown here to be entirely predictable using Goda's wave force prediction methods available from (say) 1985.

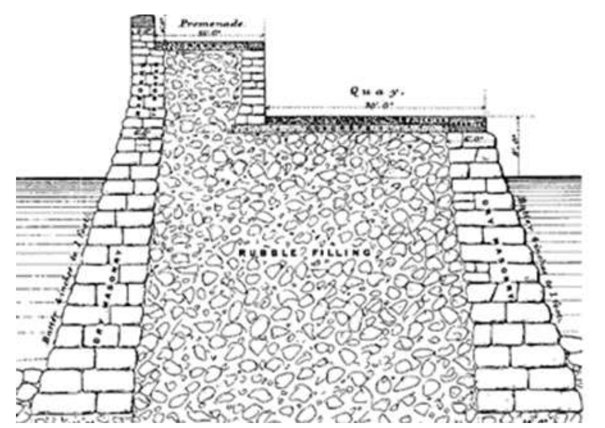
# 6 APPLICATION of ANALYSIS METHODS to SELECTED STRUCTURES

# 7 MODEL TESTS OF 'ORPHAN BREAKWATERS'

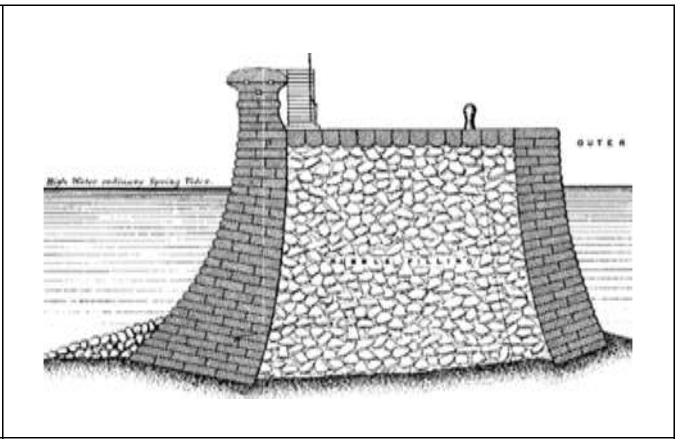
## 7.1 Background

The breakwaters of principal concern in this thesis are commonly formed by vertical or battered walls of dressed stone blocks with rubble fill, formed on rubble mounds built to low-water, as examples in Figure 7.1.

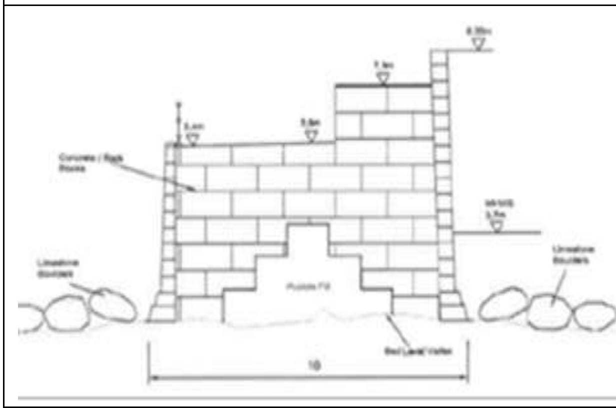
The walls are formed of hewn blocks, see discussion in Chapter 3. The core (or hearting) of the super-structures is poorly documented, probably quarry-run interspersed with broken blocks and chiselled scraps of the wall-stones.



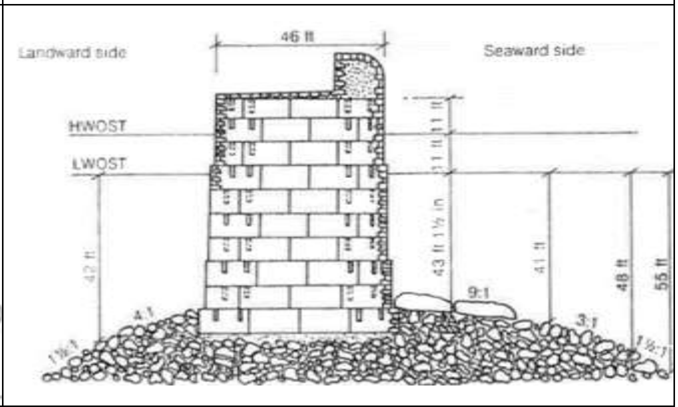

St Catherine’s breakwater, Jersey




Whitehaven, Cumbria



Heugh breakwater, Hartlepool




Peterhead, Scotland

**Figure 7.1 Blockwork walls used in assessment of block / fill proportions**

For many small harbours, maritime incomes abated 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 from which to maintain or repair their breakwater(s). In some harbours, however, landside areas have been increasingly adapted to commercial and/or residential purposes. Those new users often derive significant 'flood protection' from wave shelter given by the now orphaned breakwater(s), but seldom contribute to ‘harbour’ income available for maintenance or repair of harbour works, including the breakwater(s).

If (or when) those ‘orphan’ breakwaters fail, flood protection to areas hitherto protected from direct wave action will reduce, see for example the analysis of protection provided by the Heugh breakwater (Figure 7.2) at Hartlepool by Hampshire *et al*. (2013). Some wave reduction will still be afforded by the collapsed structure. In assessing that protection, it is important to identify the degrees of wave transmission over the collapsed structure.

**Figure 7.2 Wave attack on the Heugh breakwater at Hartlepool**
*Courtesy HRW*

Of concern to those who manage the structures, and those who benefit from their shelter, are the following questions:

a) If these structures were to fail, to what level might they be reduced?

b) How much wave protection would they provide in their collapsed form?

The tests summarised here, are intended to identify how much (or little!) wave protection will be given by the remnants of such a breakwater after its collapse.

**Table 7.1 Dimensions of example blockwork breakwaters**

| Breakwater | Section | | | Lower blocks | | | Upper blocks | | |
|---|---|---|---|---|---|---|---|---|---|
| | Height [m] | Width [m] | Block to Fill ratio, 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 |
| Peterhead | 15.9 | 14.0 | 100 : 0 | 2.0 | 2.3 | 3.9 | 2.0 | 2.3 | 4.1 |

Typical dimensions of breakwaters at Whitehaven, Blyth, Kilrush, Alderney / St. Catherine's, and Peterhead, were reviewed above. These structures are mostly vertical (or slightly battered) blockwork walls with random rubble infill. Peterhead is the exception, being almost entirely concrete blocks. Historical records contain few construction details, but nominal dimensions were extracted from photos, site visits, and old *ICE Proceedings*. Key dimensions for selected structures are summarised in Table 7.1. Core grading is seldom documented, but it is probable that the fill will be relatively evenly graded with an upper-limit diameter probably smaller than of any rubble blocks used to protect the foundation mound.

## 7.2 Design of these experiments

These model tests were intended to reproduce the general levels of wall stability, and collapse, requiring that the blocks and fill be suitably scaled. The experiments were not set-up to test any particular structure, but to give results that could be used to support generically applicable conclusions. A nominal Froude scale of 1:30 was chosen to give a test structure fitting in the wave flume available at HR Wallingford, and that could be failed by test conditions in the range of available equipment. The design of both 'dry test' and 'wet test' experiments is described by Allsop & Pearson (2016) and summarised by Allsop *et al* (2017, 2018).

The main test sections were formed by two blockwork walls founded on a trapezoidal rubble mound. The space between the block walls was filled by rubble fill, added as the blockwork walls were raised. The 'concrete' blocks used were manufactured in cement mortar to (model dimensions) 140mm long, 70mm wide, and 30mm high with a (model) density of $\rho_c$=2320 kg/m3. Correcting for the flume's fresh water, these represent blocks in seawater of 4.2m long, 2.1m wide, and 0.9m high and $\rho_c$=2400 kg/m$^3$. These dimensions correspond well with the walls at Peterhead and Blyth, but the block sizes are large compared to the other example structures. These blocks were used for both 'dry' and wave flume 'wet' testing.

The model core material was widely graded with a prototype diameter $D_{max} \approx 1.5$m, smaller than a wall block and the minimum size was $D_{min} \approx 0.15$m prototype. A set of 'angle of repose' tests described in Allsop & Pearson (2016) gave φ = 53 - 56° for the model fill.

## 7.3 Dry build experiments

The main 'wet' experiments tested the stability of different wall configurations against scaled waves up to $H_s$≈9.5m. But before those 'wet' tests, a dedicated series of 'dry' tests were conducted to provide verification testing for the wall stability model by Pearson & Allsop (2017).

The dry-build tests were designed to explore the influence of block configuration, and of core grading on the comparative stability of the walls. Most of these tests were conducted in a plywood 'box' of the same width as the wave flume, open at each end and at the top, but braced side-to-side. Later tests used a plywood back-wall to form one 'end', so these tests only tested a single wall. The objective of both series of tests was to explore the influence on wall stability of block orientation, wall construction (e.g. bonded vs. columnar), and of fill grading, At its simplest, the tests assessed how high the wall and fill could be taken before the wall fell over. The block configuration is given (in prototype metres at a scale of 1:30) in Table 7.2 below as are dimensions across the test section; upwards; and into the section.

**Table 7.2 Test section configurations for 'dry build' tests**

| Build number | Block config., L x H x W (m) | Bond | Side details | No. rows | Fill height (m) | Comments |
|---|---|---|---|---|---|---|
| A1 | 4.2 x 0.9 x 2.1 | Columnar | 2 walls, timber | 21 | 17.25 | |
| A2 | 4.2 x 0.9 x 2.1 | Bonded | 2 walls, timber | 22 | 15 | |
| A3 | 4.2 x 0.9 x 2.1 | Columnar | 2 walls, timber | 18 | 15 | |
| A4 | 4.2 x 0.9 x 2.1 | Columnar | 2 walls, timber, PVC | 15 | 13.5 | Less arching, |
| A5.1 | 4.2 x 2.1 x 0.9 | Columnar | 1 wall, timber, PVC | 3 | 6.3 | |
| A5.2 | 4.2 x 2.1 x 0.9 | Columnar | 1 wall, timber, PVC | 3 | 5.4 | |
| A5.3 | 4.2 x 2.1 x 0.9 | Columnar | 1 wall, timber, PVC | 3 | 5.8 | |
| A6.1 | 4.2 x 2.1 x 0.9 | Bonded | 1 wall, timber, PVC | 3 | 6.0 | |
| A6.2 | 4.2 x 2.1 x 0.9 | Bonded | 1 wall, timber, PVC | 3 | 5.4 | |
| A7 | 2.1 x 4.2 x 0.9 | Columnar | 1 wall, timber, PVC | 1 | 4.2 | |
| A8 | 4.2 x 2.1 x 0.9 | Columnar | 1 wall, timber, PVC | 3 | 7.5 | Wet core |
| A9 | 4.2 x 2.1 x 0.9 | Bonded | 1 wall, timber, PVC | 3 | 7.5 | Wetted core |
| A10 | 4.2 x 0.9 x 2.1 | Columnar | 1 wall, timber, PVC | 12 | 8.1 | Second core |

The dry-build walls 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 Pearson & Allsop's (2017) spreadsheet model; 20 rows high at failure in the dry-builds versus 11 or 12 in the model. This increased stability was attributed largely to jamming, or arching against the flume walls.

The single clearest finding from these construction tests is that the resistance of the model wall to toppling can be significantly influenced by transverse (inter-block) forces, primarily from arching across the width of the test section. Later tests therefore tried to reduce friction against the side wall, thereby reducing their restraining effects. Without such distortions, dry-build failures were generally brittle, initiated by the horizontal component of the inter-granular forces within the core. Failure of the ~30m long section (across the test flume / box) by overturning was often resisted by arching of the top rows of the wall (of the order of 1-1.5m horizontal displacement at centre of crest). As the effect of arching increases, the wall essentially 'unzips' or 'spreads' along a vertical line down the centre. This lengthens each course. Eventually, a course will arch so much that 'snap-through buckling' reduces pressure between blocks and the flume walls. Using 'slip plates' against the 'flume' walls (see example in Figure 7.3) generally reduced the friction, removing some (but not all) of this artificial resistance to failure. The concrete blocks are not perfectly uniform, so columnar placement still resulted in some interlock between adjacent columns, and hence sideways transfer of loadings / reactions.

**Figure 7.3 Example 'dry-build' wall**
*Before failure, with PVC 'slip' panels*

**Figure 7.4 Example dry-build failure**
*Left - immediately after collapse. Right - failed structure with spilled core removed to reveal arched (but still intact) wall toe*

A typical dry-build failure is shown in the left-frame of Figure 7.4 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. It shows that the wall overturned at a hinge not at the bottom row, but several rows up. This form of failure was repeated in the wave flume experiments later, and was identified in the historic failure of the breakwater at Skateraw in Chapter 2.

The primary results of the dry-build tests were the wall heights reached before failure, as detailed in Table 7.2. The purpose of these tests was mainly to assist design the 'wet' test sections, but also to provide validation to Pearson's simplified stability model described by Pearson & Allsop (2017).

## 7.4 Wave flume tests

### 7.4.1 Test set-up

The geometries for the six test sections tested in the wave flume (wet) tests are summarised in Table 7.3. For each model breakwater, the wall was formed by two single-skin walls using blocks 4.2m longitudinally (parallel to the breakwater centreline), 2.1m deep (into the section, and 0.9m high. Test Series 1 used blocks laid bonded (offset from one layer to the next). The later walls were formed by blocks laid in simple columns in an effort to reduce restraint by arching (partially successful).

**Table 7.3 Test section configurations for wave flume tests**

| Test series | Block config., L x H x W (m) | Bond | Side details | No. rows | Height (m) | Comments |
|---|---|---|---|---|---|---|
| 1 | 4.2 x 0.9 x 2.1 | Bonded | Flume walls | 16 | 15.80 | |
| 2 | 4.2 x 0.9 x 2.1 | Bonded | Flume walls | 18 | 17.45 | |
| 3 | 4.2 x 0.9 x 2.1 | Columnar | PVC slip plate | 16 | 16.23 | Crest paved |
| 4 | 4.2 x 0.9 x 2.1 | Columnar | PVC slip plate | 20 | 18.82 | |
| 5 | 4.2 x 0.9 x 2.1 | Columnar | PVC slip plate | 11+2 | 12.16 | Slabs on crest, 2 rows in parapet wall. |
| 6 | 4.2 x 0.9 x 2.1 | Columnar | PVC slip plate | 11+2 | 12.21 | As Series 6, finer core |

Each test section was constructed on a rubble mound from the flume floor at -24m (the intended test water level being at 0m) up to a mound platform level intended to be at -9m, but which had generally settled to about -10m. The build process was to build the foundation mound from roughly graded rock, tamped down and flattened, see Figure 7.5. The mound shoulders were covered by plywood during wall construction to protect the mound.

**Figure 7.5 Rubble mound platform before wall construction**

A finer (bedding) material was used to regulate the mound where the model walls were to be built giving a base width (between outer faces) of 0.815m model (24.45m). The walls were progressively built course by course with the fill poured in gently every two courses to prevent any sudden pressure changes within the model.

The composition of each of the models in Series 1-6 is summarised in Table 7.5. In each instance the wall height is calculated from surveys of the pre-construction foundation and completed crest levels. The volumes of crest protection and parapet walls are then added where appropriate to give the total section volume above the platform (per m width).

It is noted in Table 7.5 that there was always a (relatively small) upwards ‘creep’ as the net wall height exceeded the simple product of the number of rows x the nominal block height (0.9m). This upward 'creep' was greatest for the two sections where the blocks were laid bonded rather than columnar.

**Figure 7.6 Wall construction**

**Table 7.4 Test section compositions for wave flume tests**

| Test series | Foundation level (m) | Wall crest level (m) | Wall height (m) | Volume ($m^3$/m) | Upward creep (m) | Upward creep (5) |
|---|---|---|---|---|---|---|
| 1 | -8.52 | 7.28 | 15.80 | 16 | 386 | 1.40 |
| 2 | -9.30 | 8.15 | 17.45 | 18 | 426 | 1.25 |
| 3 | -10.43 | 5.80 | 16.23 | 16+1 | 412 | 0.93 |
| 4 | -9.24 | 9.59 | 18.82 | 20 | 460 | 0.82 |
| 5 | -7.77 | 4.39 | 12.16 | 11+2 | 312 | 0.46 |
| 6 | -7.84 | 4.37 | 12.21 | 11+2 | 312 | 0.51 |

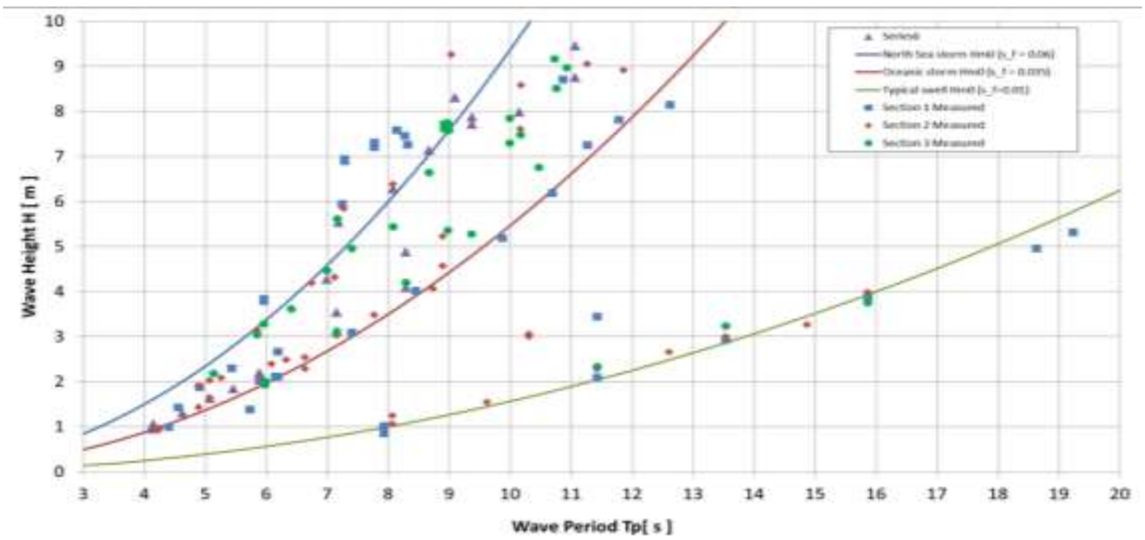



**Figure 7.7 Wave conditions for conditions A, B, and C (at 1:30 scale)**

Tests used three wave steepnesses shown in Figure 7.7. Conditions A ($s \approx 0.06$) represented North Sea storms generated by high winds. The waves are steep with high energy density. These conditions were generally used to push the structure to failure.

Conditions B ($s \approx 0.035$) represented seas from oceanic storm, as in the Atlantic, of medium steepness and relatively high energy density. These conditions were likely to influence reshaping of the collapsed breakwater and (probably) give increased wave transmission.

Conditions C represented typical swell waves of low steepness ($s \approx 0.01$) and lower energy densities. Wave periods were longer, potentially causing greater wave transmission. These conditions most closely represent operability in protected areas behind a breakwater.

In each series, wave conditions started at a low or moderate wave height, and were stepped up in each subsequent test part. Most test parts were run for 500 (a few for 1000) waves. The shorter duration was generally used for the initial wall failure tests using condition A.

Wave conditions were measured by wave gauges (WG) in front of the seaward wall of the breakwater, see layout in Figure 7.8. Incident and reflected spectra were calculated using a 4 gauge array with inter-gauge spacings set to cover the range of frequencies, using the least squares methods by Mansard & Funke (1980) based on the analysis approach of Isaacson (1991) to extract incident and reflected significant wave heights, $H_{si}$ and $H_{sr}$, the peak period $T_p$ and spectral period $T_{m-1,0}$. Reflection coefficients $C_r$ were calculated from the ratio of reflected to incident wave heights.

Transmitted waves were measured at a gauge behind the breakwater seaward face, so approximately 75m landward from the back wall of the breakwater. The main wave parameters extracted were the transmitted wave height, $H_{st}$, peak period $T_p$ and spectral period $T_{m-1,0}$. The transmission coefficient $C_t$ was calculated from the ratio of transmitted to incident wave heights, $C_t = H_{st} / H_{si}$.

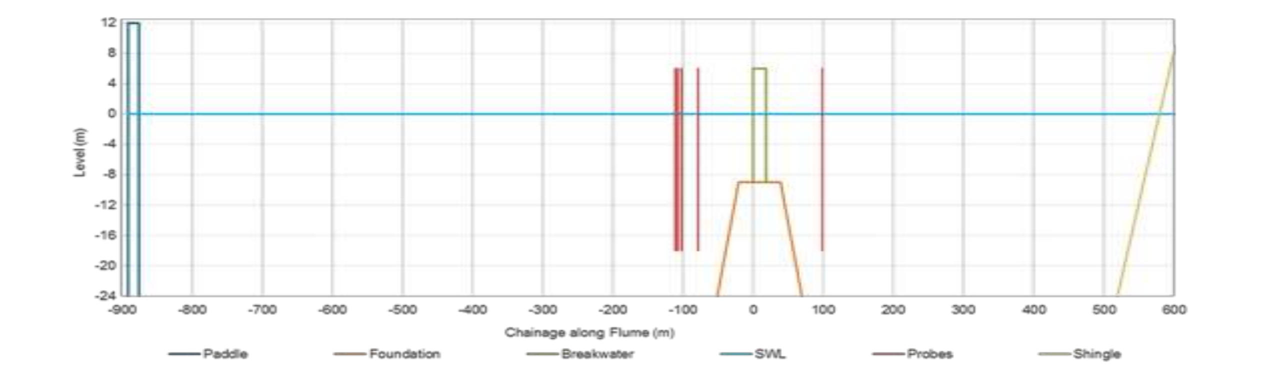



**Figure 7.8 Schematic of wave flume**
*Showing model location, wave gauges for incident, reflected and transmitted waves*

Profiles of the breakwater sections (before, during and after collapse) were measured using a 'dipping' profiler with a touch-sensitive head fitted with a hemi-spherical head, sized to fit the general size of the rubble core. The measurement device drove itself along a longitudinal beam in steps equivalent to 0.6m, 'dipping' down onto the section to be measured. Three longitudinal profiles were spaced 7.5m (250mm) apart across the flume. Each profile covered a horizontal distance equivalent to just under 60m.

### 7.4.2 Test Series 1

This first wall was constructed 16 blocks high, both front and back, bonded pattern. To limit mixing of the wall fill with the foundation mound, a geotextile was laid over the mound, but this proved of limited utility and was omitted in later tests.

Test parts (runs) 1-14 were run to achieve wall collapse, and then test parts (runs) 15-36 were used to measure wave transmission, see test evolution shown in Figure 7.9 Conditions for each test part are tabulated in Appendix B.

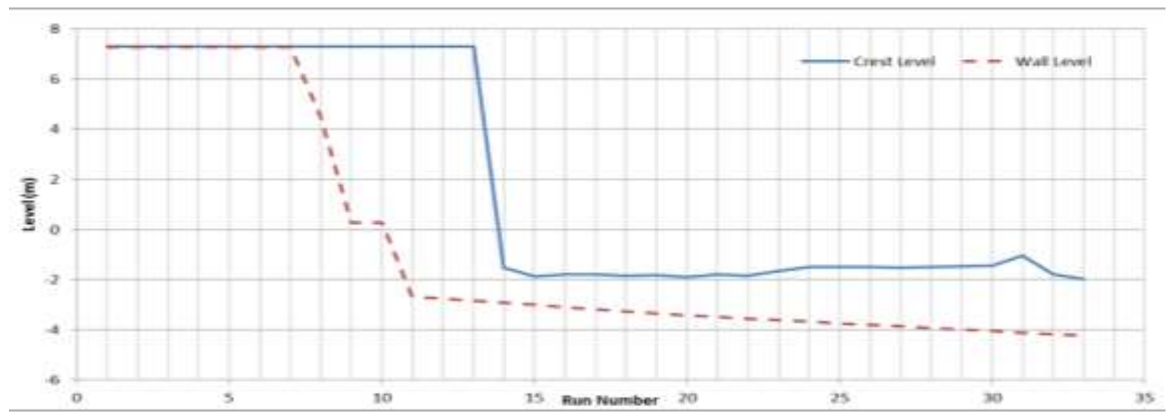



**Figure 7.9 Crest height evolution for Test Series 1**

**Figure 7.10 Test Series 1, post front wall failure**

Test wave heights were increased along the A steepness curve in Figure 7.7 starting at $H_s$=1.4m. The seaward wall began to fail at around $H_s$=7.2m (Run 9) when the crest course was knocked off, pushing blocks below backwards. Uplift velocities lifted crest blocks out and off the front wall, leading to increased overtopping, in turn washing out fill material.

Multiple blocks then fell seawards from the front face as overtopping and local wave pressures causing 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. As more blocks were extracted, more overtopping percolated into the fill, forcing out further blocks. Once the seaward wall had failed, wave backwash washed fill out until the seaward blocks and rubble fill were graded by wave action to a relatively shallow slope down the front of the structure. By the end of the test the remaining blocks in the seaward wall were covered by fill.

With the seaward wall down, the waves mainly broke onto the new rubble beach, attacking the rear wall with significantly less force. Even so, the rear wall showed considerable arching, probably leading to the wall staying upright longer than it would without lateral restraint.

Following the collapse, the test wave conditions were changed to the 'persistent' and 'swell' conditions, starting at substantially lower wave heights, typically $H_s \approx 0.8$ - 1.0m. During most of the wave transmission tests using conditions B or C, changes to the (marginally-submerged) profile were relatively small with most mobile material passing backwards over the remains of the rear wall. Overall, rock movement flattened and smoothed the submerged profile as loose rocks and blocks rolled seaward or landward.

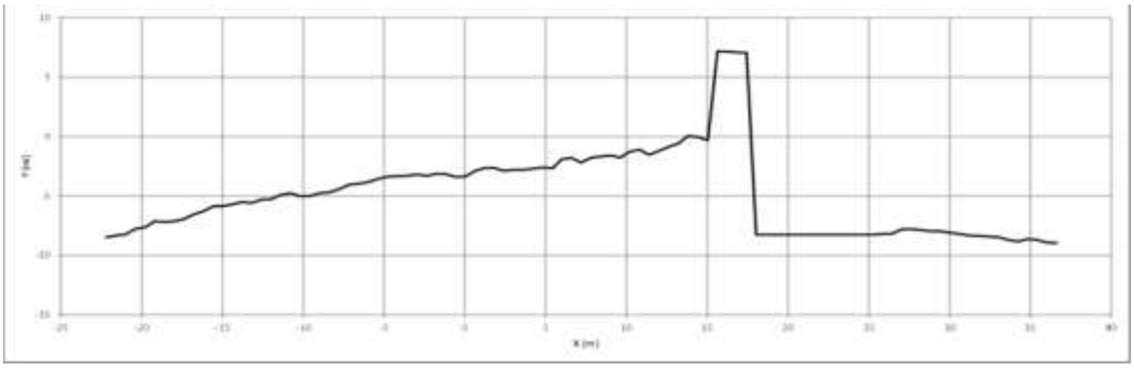


Figure 7.11 Test Series 1, profile taken post seaward wall failure, run 12

### 7.4.3 Test Series 2

The second wall was constructed 18 blocks high on the front wall, but fill and back wall were taken only to 16 courses, giving a parapet wall. Blocks were again laid in bond. The geotextile used in Series 1 was omitted as it had made no useful contribution. Sixteen test parts (each 500 waves) were required to fail the structure in Series 2 (failure occurring over test parts 48-52). Then test parts 53 to 79 were used for wave transmission measurements. Test conditions for each test part are listed in Appendix B.

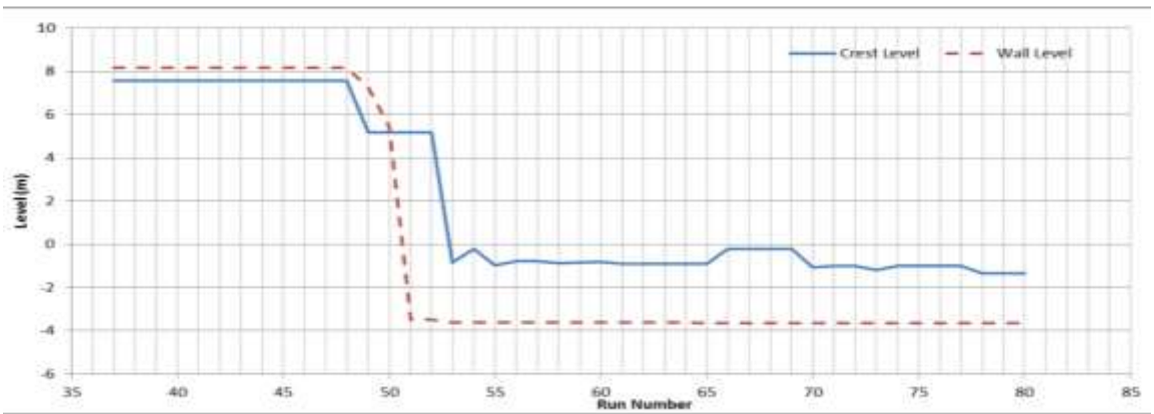



Figure 7.12 Crest height evolution for Test Series 2

Again, wave conditions increased along the 'storm' steepness curve until at $H_s$=8.6m (Run 48) blocks on the top course were flipped back. As the test progressed, 3 blocks projected relative to those beneath, allowing upward flows to lift their protruding edge and flip them backwards and out of the structure, as seen in Series 1. Once this key group of blocks had been removed, waves were able to scour out the remaining blocks and fill from inside the breakwater. This process soon exposed the leeward wall to direct wave impact, this leading to an increased rate of scour causing the section to form a profile similar to that seen in Series 1.

Multiple blocks were then extracted seawards from the front face as overtopping and local wave pressures caused multiple gaps in the blockwork at high level. Some courses resisted further due to arching against the flume sides. As more blocks were extracted, more overtopping penetrated into the rubble fill, forcing out further blocks. Once the seaward wall had failed, 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. By the end of the test the remaining blocks in the seaward wall were covered by fill, Figure 7.13

**Figure 7.13 Series 2, post collapse, view from seaward**

Following the front wall collapse, waves had direct impact on the leeward wall than in Series 1, in part due to the longer period waves, giving more impulsive bores over the block / fill beach formed by the collapsed front face. Arching once again meant that the wall stayed intact for longer than would be likely in prototype structures of any significant length. In the model, failure of the wall was therefore relatively slow, perhaps less brittle than in reality.

**Figure 7.14 Series 2, post collapse, view from leeward**

Once the walls had collapsed, profile changes for Series 2 were minor as the collapsed mound was stable under the (smaller) transmission test waves. Example profile graphs from the dipping profiler can be seen in Figure 7.15.

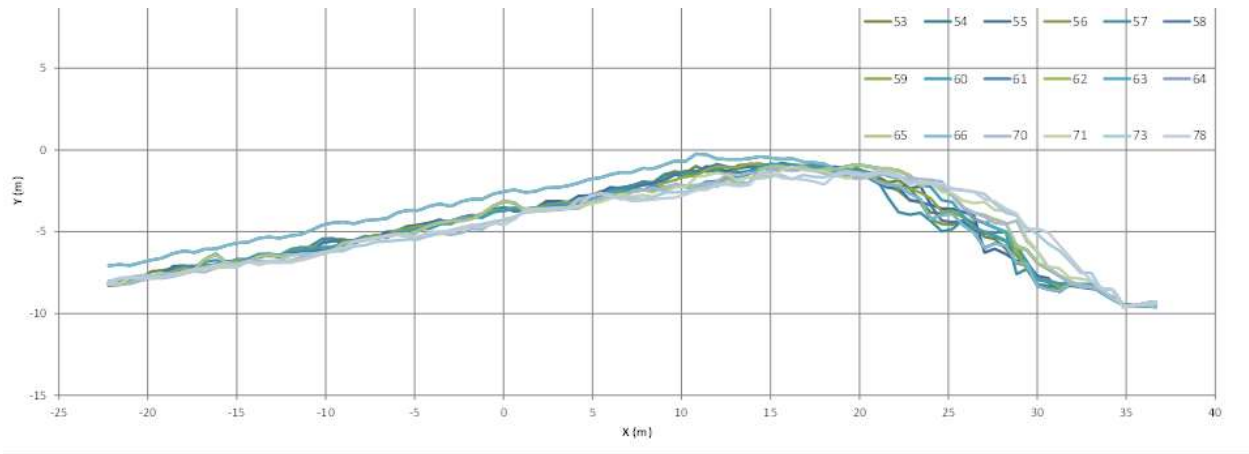



Figure 7.15 Test Series 2, post collapse profiles, runs 53 - 78

Following completion of Test Series 2, the collapsed wall / mound was excavated. The remains of the front wall appeared to show voiding under the wall, perhaps by scour, leading to wash-out of some lower blocks.

Figure 7.16 Excavated front wall, note vertical arching where foundation has scoured

### 7.4.4 Test Series 3

The Series 3 wall was built to 15 blocks high unbonded (columnar) following dry build A1, filled with standard fill. A layer of fines on top of the fill allowed placing of blockwork 'capping' bringing the section height to 16 blocks, see Figure 7.17.

Figure 7.17 Crest protection for Series 3, a) bedding layer for crest; b) crest blocks

Figure 7.18 Extraction of front face block in Series 3

The seaward wall initially experienced a single block extraction Figure 7.18, showing that even with columnar placement, load transfer between columns can leave individual blocks un-restrained. With the smooth crest, the Series 3 wall transmitted more wave energy over the structure. At $H_s$=7.05m, the seaward edge of the capping layer however began to lift by up-rushing waves. This lifting and dropping caused the crest cap to deform. This movement continued until the row had been thrown backwards or washed forwards, creating a gap in the front of the wall, Figure 7.19.

Figure 7.19 Damage in Series 3 to a) front face, b) rear face

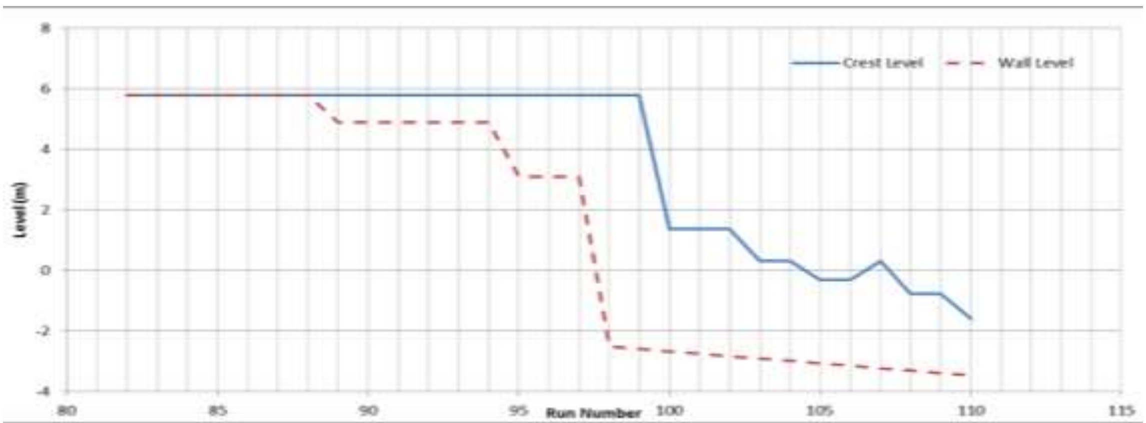



Figure 7.20 Crest height evolution for Test Series 3

Once this gap had been opened, waves penetrated under the cap, washing individual blocks off relatively quickly. This then allowed the waves to pull the fill and front wall down within 500 waves, forming a profile similar to that seen in Series 1 and 2. The evolution of damage to the wall through Series 3 is summarised in Figure 7.20.

On the leeward side, the rear wall in Series 3 showed little movement until the front wall fell, unlike the previous tests where blocks had been extracted from the rear wall even before the front wall failure. This was probably due to a higher proportion of wave energy being transmitted by overtopping rather than driving internal pressures in the granular fill.

With fill at the rear wall scoured out by overtopping, the rear wall fell under $H_s$=8.1m and $T_p$=11.2s, The failure was however probably delayed by arching restraint. A consequence of the delay before rear wall failure was that relatively more fill had been washed out seaward, so less was present in the post-collapse leeward slope.

### 7.4.5 Test Series 4

Series 4 was built to 20 blocks with fill all the way up, blocks in columnar pattern. PVC side sheets were used, but even with this attempt to reduce the arching restraint, the section was successfully constructed to 20 blocks height. Perhaps the flume walls were affording more arching restraint than the dry-build rig.

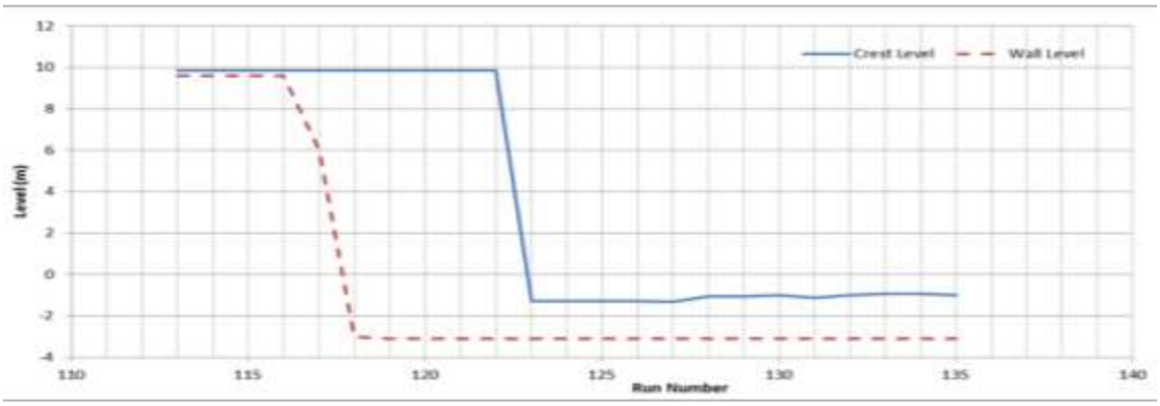



**Figure 7.21 Crest height evolution for Test Series 4**

At $H_s$=6m, the top layer began to show signs of movement along the flume, with some of the blocks getting pushed backward and one in the second row being extracted forwards, Figure 7.22. As before, the full list of test conditions is given in Appendix B.

**Figure 7.22 Series 4, start of damage with sink-hole in fill following loss of front face blocks**

Soon after, more blocks were extracted from the central columns, allowing fill to be extracted manifesting itself as a sinkhole against the inside of the front wall. Once enough blocks had been extracted, a section of the wall began to lift and tilt backwards due to the water pushing it from underneath, some of these blocks then fell down into the gaps where the row below had been extracted, Figure 7.23 This drop in height of the wall allowed the incident waves to overtop the wall and wash loose blocks and fill down from the section. Figure 7.24 shows the seaward wall and the blocks in front of it once they had fallen. The following tests ($H_s$=8m) steadily brought the front wall down course by course, washing the fill out with it.

**Figure 7.23 Series 4, progressive damage to front wall following partial loss of fill**

The leeward wall failed in a more brittle manner and earlier in Series 4 than Series 1-3, perhaps because of reduced arching given by the PVC slip plates. More core was washed over into the rear face when it collapsed. The rear face also collapsed more monolithically in Series 4 than in Series 3, Figure 7.25.

**Figure 7.24 Rear face following failure of front face, Series 4**

**Figure 7.25 Collapsed rear face, Series 4, note close spacing of the blocks**

### 7.4.6 Test Series 5

The design of Series 5 was significantly informed by the 'dry build' tests described earlier. These identified that the resistance to collapse of earlier test sections had been influenced by 'jamming' of blocks against the flume walls, an unwanted model effect. Following the lessons from the dry 'build' tests, double strips of (slippery) PVC sheet were hung against the flume walls to give 'high-slip' against the walls, and thus to reduce the (over-)stable wall heights. The walls in Series 5 were therefore constructed to only 11 blocks height, giving a crest level at +4.4m, and a wall width of 24.4m.

Following the experience of Series 3, it was anticipated that a reduced crest level would lead to much increased overtopping. Two measures were taken to resist overtopping causing excessive early erosion. As with most of these types of breakwater, the crest itself was protected by large slabs, in this instance simulated by aluminium sheet (ρ ~2.4t/m3) of thickness equivalent to 0.1m and plan of 3m x 3m, Figure 7.26. Even so, it would be expected that the leading edge of such slabs could easily be lifted by overtopping flows. So, as was often done in breakwaters of the era considered, a parapet wall was added along the seaward edge, formed here by two rows of wall blocks placed end-on.

**Figure 7.26 Test section for Series 5, showing crest slabs and parapet wall**

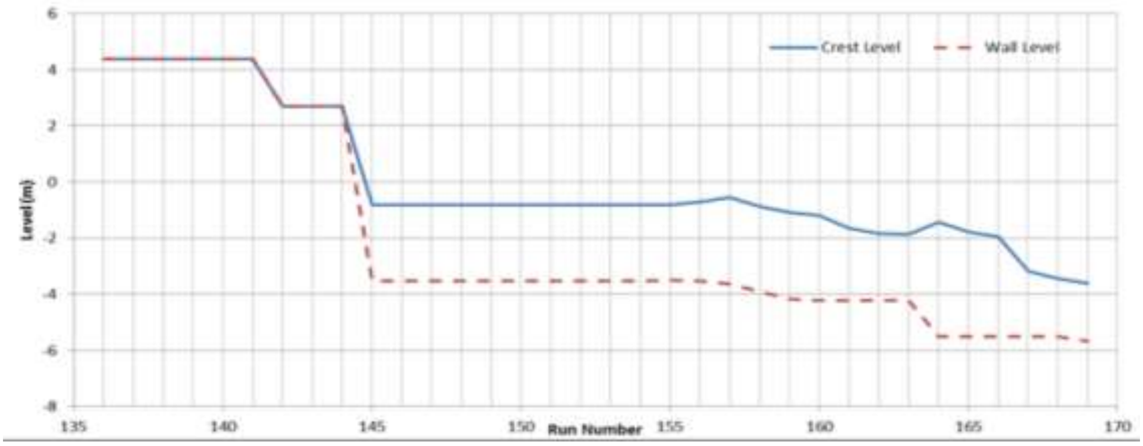



**Figure 7.27 Process of collapse Test Series 5**

The last change for Series 5 was that the test water level was reduced to SWL = -1.5m, again to reduce the proportion of incident wave energy lost by overtopping.

As previously, the test wave conditions were increased along the 'storm' steepness curve ($s \approx 0.06$), Figure 7.27, until Test part 138 when as the waves reached $H_s$=4.3m some of the parapet wall blocks started to slide backwards. During Test part 139 at $H_s$=4.9m more crest blocks were pushed backwards, thus allowing wave forces to lift the blocks above them, Figure 7.28

**Figure 7.28 Onset of damage during Tests 138 and 139**

During Test 141 ($H_s$=5.8m) one half of the seaward wall collapsed around the hole left by the two blocks that had been extracted in the previous test parts. During Test 142 (at essentially the same wave condition) the remaining half of the front wall collapsed suddenly. The majority of the capping slabs were swept over the crest onto the leeward foundation, Figure 7.29. Any capping slabs that became wedged in the rubble mound were removed manually, judging that their prototype equivalents would have failed structurally.

**Figure 7.29 Section 5 after collapse of the seaward wall**

By this stage, the rear wall had again arched against the flume wall, again artificially restraining the wall blocks. The double slip wall sheets were adjusted, and some of the wall blocks were moved slightly to reduce this effect. The rear wall collapsed during test parts 143 / 145.

An initial set of transmission and reshaping tests (Runs 145-154) were run with low wave heights at SWL=-1.5m. During these tests there was no significant rock displacement or profile reshaping until Run 155 at $H_s$ = 3.8m, $T_p$ = 8.7s. Plunging waves pushed core material upward (cf. Graauw, 2013), temporarily raising the crest level. This continued in Tests 156 and 157, after which the waves broke earlier on the mound and many up-rushing bores washed rocks off the rear of the crest. The final set of tests (at water level of SWL = 0.0m were run to quantify transmission without further reshaping. Wave conditions and water levels are tabulated in Appendix B.

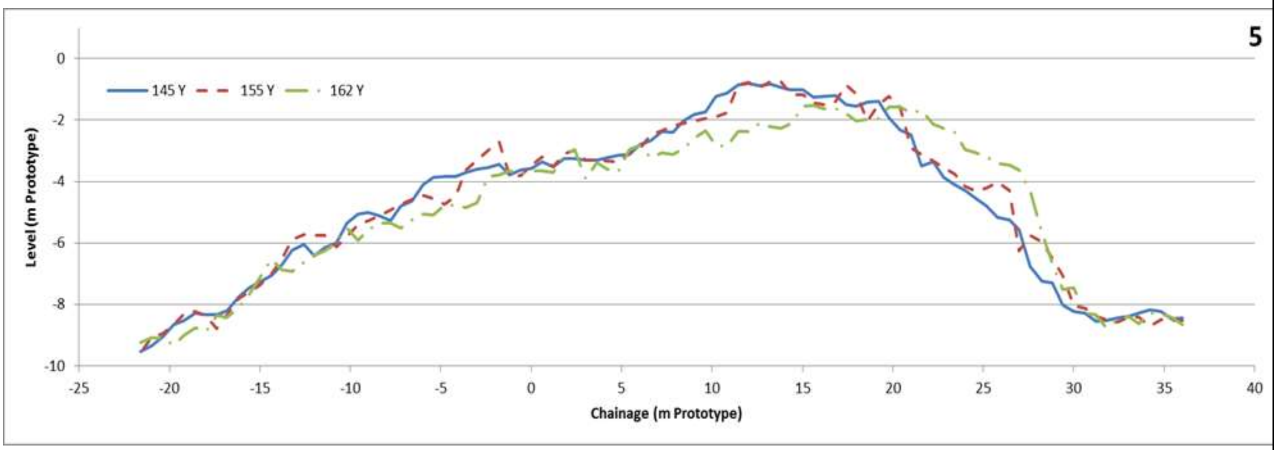



**Figure 7.30 Collapsed profiles, Series 5**

### 7.4.7 Test Series 6

The design of this test section followed the revised dry-build test A4 with a wall reaching 15 rows high. That dry test had suggested that reducing wall-friction would also reduce the stable wall height under wave action. The wall height for Series 6 was therefore reduced to 11 rows. But this low crest would however allow substantially greater overtopping with the potential to remove crest blocks and fail the rear wall. To reduce the onset of this failure mode, the section was protected across the crest using 90mm (3mm model) thick aluminium plates (3m x 3m) simulating concrete slabbing, as for Series 5. As in most historical breakwaters, this crest slabbing was then surmounted by blocks forming a parapet wall, further reducing overtopping flows. Another change made for this last series was to reduce the median size of the core material by adding an increased proportion of finer material.

**Figure 7.31 Series 6, block extraction on the front face, later damage progression**

During Test Series 6, waves were increased from $H_s$ = 2m to $H_s$=5.7m when the first block was extracted from the front face, and capping material began to be washed off the structure. As damage at this test condition was seen to be continuing, the same conditions was re-run, steadily extracting more blocks, particularly in the rows just below the capping layer. During Run 173 a row of blocks had been extracted and the crest wall on top pushed back such that it was resting mainly on the rock material and capping layer, rather than on the main front wall blocks.

Wave impacts also pulled blocks out seaward from the wall below, causing a large hole to appear in the centre of the section. This collapsed quite quickly, and the front wall collapsed in test part 177.

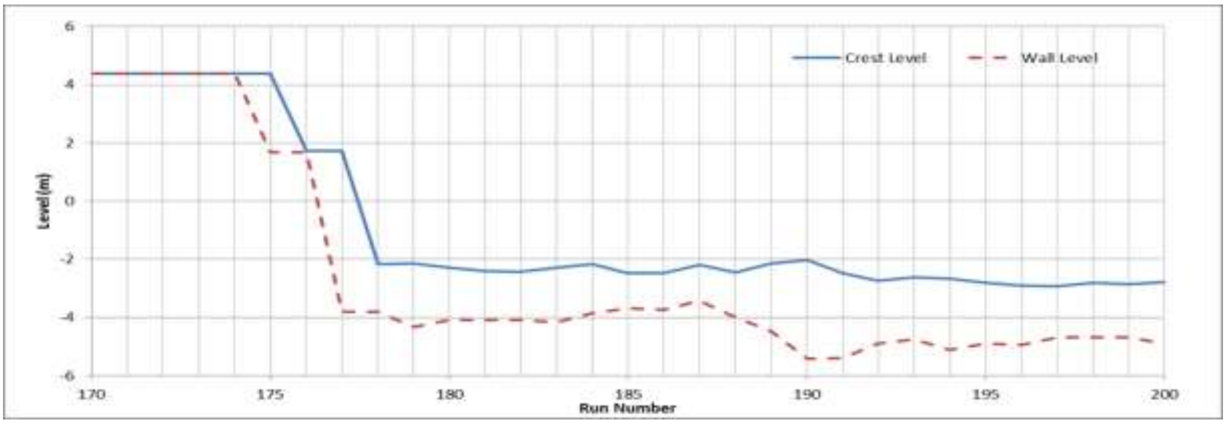



**Figure 7.32 Process of collapse Test Series 6**

The leeward wall failed faster than in previous tests, probably due to the reduced arching effect from the double slip panels on each side. This process started with the top layers of blocks being pushed backward and then the core infilling around them to form a fairly stable slope.

**Figure 7.33 Series 6, leeward wall post failure**

During the transmission tests the mound did not transform very significantly. There was a steady (but slow) crest height recession through the tests, occasionally a wall block or large rock would be carried up the slope appearing to cause a temporary rise in crest height, influencing the average profile even when averaged across three profile lines, Figure 7.34

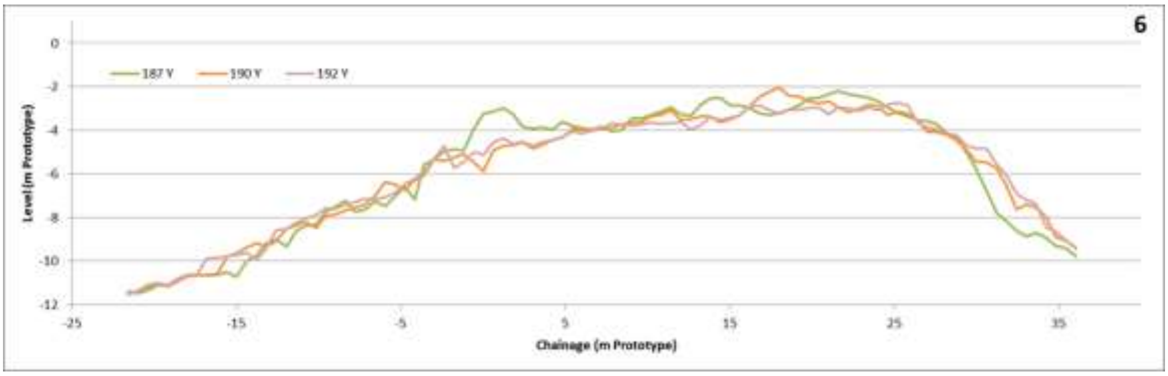



**Figure 7.34 Collapsed profiles, Series 6**

## 7.5 Wave transmission

Each Test Series used wave conditions following the three wave steepnesses, but the 'stability tests' described in section 7.4 primarily used the steepest conditions, increasing the wave height rising up the high steepness curve, A. Wave gauges on the seaward side of the test section quantified reflections from the test section, while another wave gauge behind the breakwater measured transmitted waves. Reflected and transmitted (significant) wave heights were then divided by the incident (significant) wave height to give reflection and transmission coefficients $C_r$ and $C_t$ respectively. These have been plotted against each test number, series by series, see *e.g.* Figure 7.35. Such a simplistic presentation (omitting information on wave height or period) gives a useful indicative 'history' of each test, and changes in reflections and transmission may be correlated with the section profile plots shown in section 7.4. The reflection coefficient is relatively little influenced by wave height *per se*, but will be reduced for larger waves as they start to transmit over the breakwater. Conversely, wave transmission over the (partially) collapsed structure, being a markedly less 'linear' process, will tend to be greater for larger waves.

### 7.5.1 Test Series 1

For Test Series 1, reflections (as in Figure 7.35) suggest that damage to the front wall might have started relatively early. This is slightly at variance with the profile plot shown earlier in Figure 7.9 suggesting that damage only really changed the wall in about Test Part 8. This certainly matches the reduction in reflections as the vertical face becomes a rock slope.

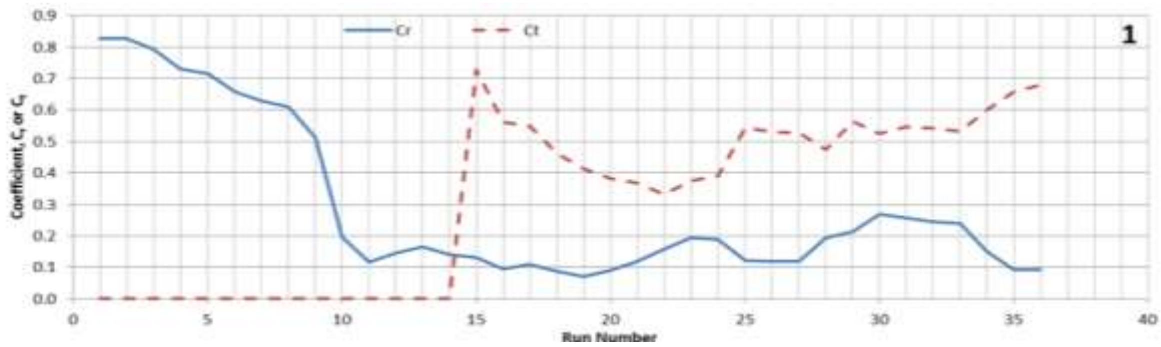



**Figure 7.35 Series 1, reflections and transmission**

Wave transmission is however relatively little influenced by damage to the front wall, depending critically on the integrity or failure of the rear wall. The sudden increase of transmission in Test Part 14 therefore correlates well with wall failure. Thereafter, reflections continue at a relatively low level, $C_r < 0.25$, whilst transmission stabilises at $C_t \sim 0.55$.

### 7.5.2 Test Series 2

Reflections and transmission are similarly plotted for Test Series 2, Figure 7.36. Here the process of wall failure is more abrupt with the delay between collapse of the front and rear walls being shorter. This is shown in the increase in transmissions around Test Part 52 following soon after the rapid fall in reflections as the rubble slope starts to develop from the debris of the front wall collapse in Test Part 49.

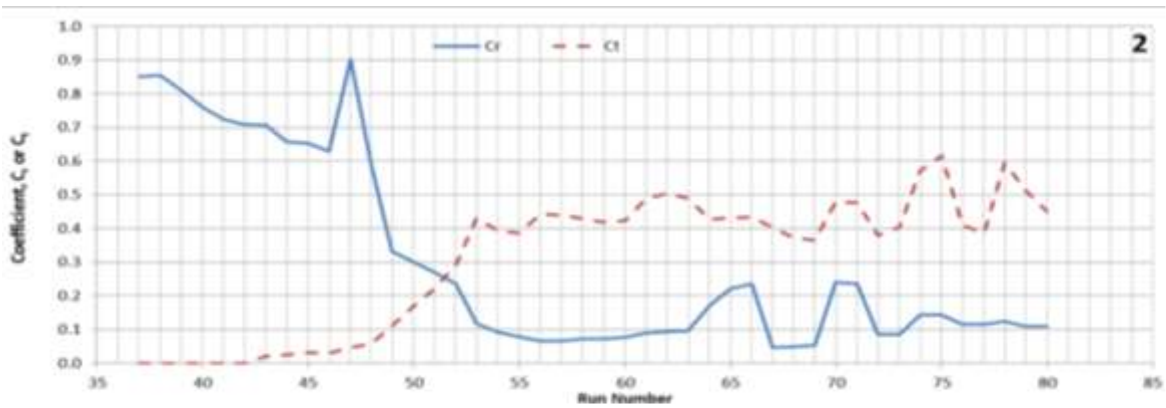



**Figure 7.36 Series 2, reflections and transmission**

### 7.5.3 Test Series 3

The process of collapse in Test Series 3 (see Figure 7.37) is very much more gradual with reflections reducing as wave heights increase and overtop more. This is then shown in the steady increase in transmitted wave height by overtopping up to Test Parts 97-99. The rear wall then fails incrementally up to Test Part 105. Thereafter transmission increases as reflections reduce.

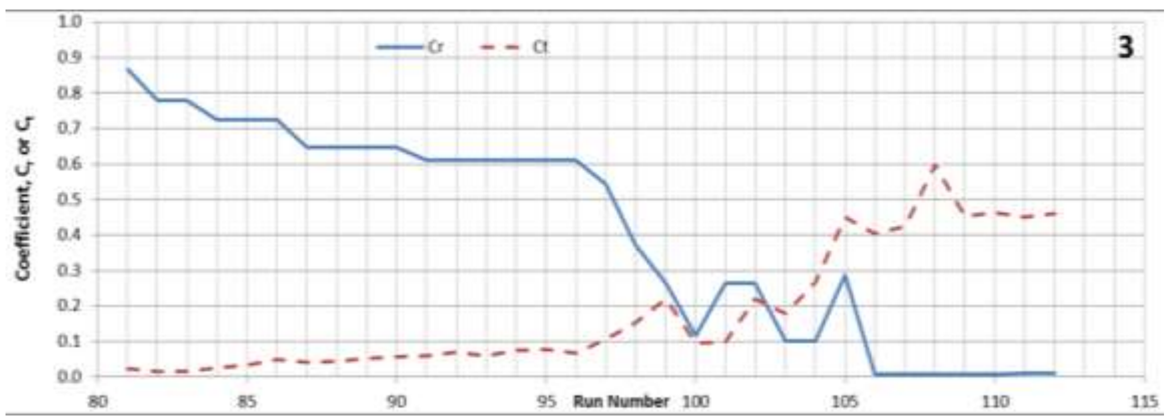



Figure 7.37 Series 3, reflections and transmission

### 7.5.4 Test Series 4, 5 and 6

This process was repeated in Series 4 (Figure 7.38) with a similar lag in the reduction of reflections as the front wall fails before the rear wall.

As expected, Series 5 (Figure 7.39) and Series 6 (Figure 7.40) behaved similarly. Reflections fell rapidly with increased overtopping, then as the vertical front face degraded to a slope. Again the increase of transmission was delayed until the rear wall collapsed, then transmission varied with the test wave condition.

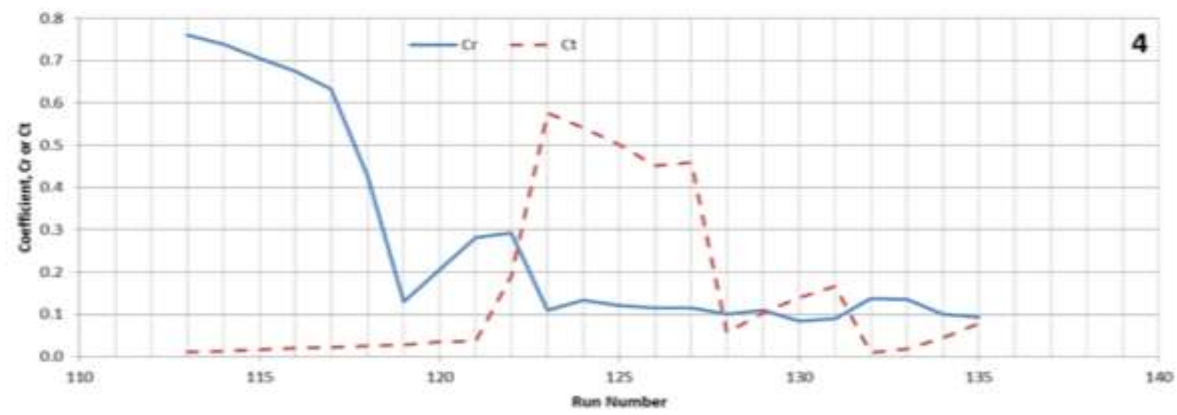



Figure 7.38 Series 4, reflections and transmission

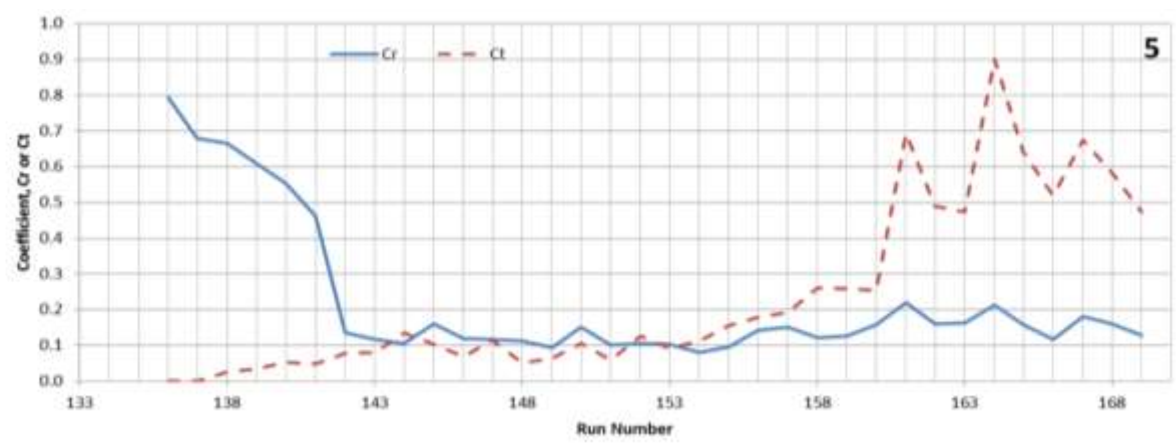



Figure 7.39 Series 5, reflections and transmission

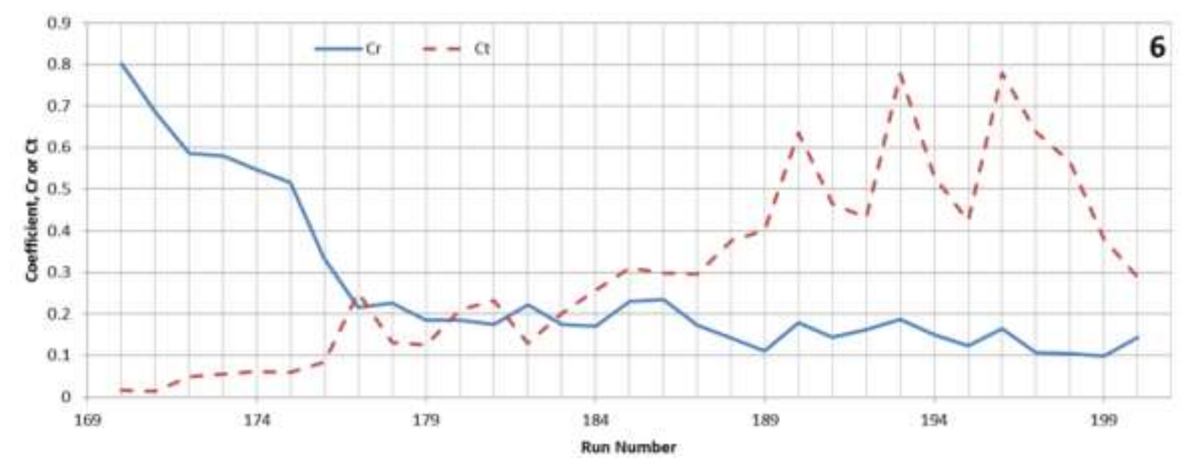



**Figure 7.40** **Series 6, reflections and transmission**

### 7.5.5 Discussion on wave transmission

**Effect of wave height**

Transmitted waves have been compared with incident wave heights to derive the coefficient of wave transmission, $C_t = H_{st} / H_{si}$. In turn, $C_t$ has been compared with the dimensionless freeboard, $R_c/H_{si}$, Figure 7.41. This form of presentation, using only the wave height $H_s$ omits any effect of wave steepness. It can be seen however that clouds of measurements at different wave steepnesses (conditions A vs B vs C) are indeed plotting separately. An attempt was therefore made to explore the utility of an alternative presentation from Report SR 57 by Powell & Allsop (1985) which includes wave steepness, where $C_t$ is plotted against $R^* = (R_c/H_{si})/(\sqrt{s(2\pi)}$ in Figure 7.42. The improvement was however very slight, probably not worth the increase of complexity in the plotting parameter.

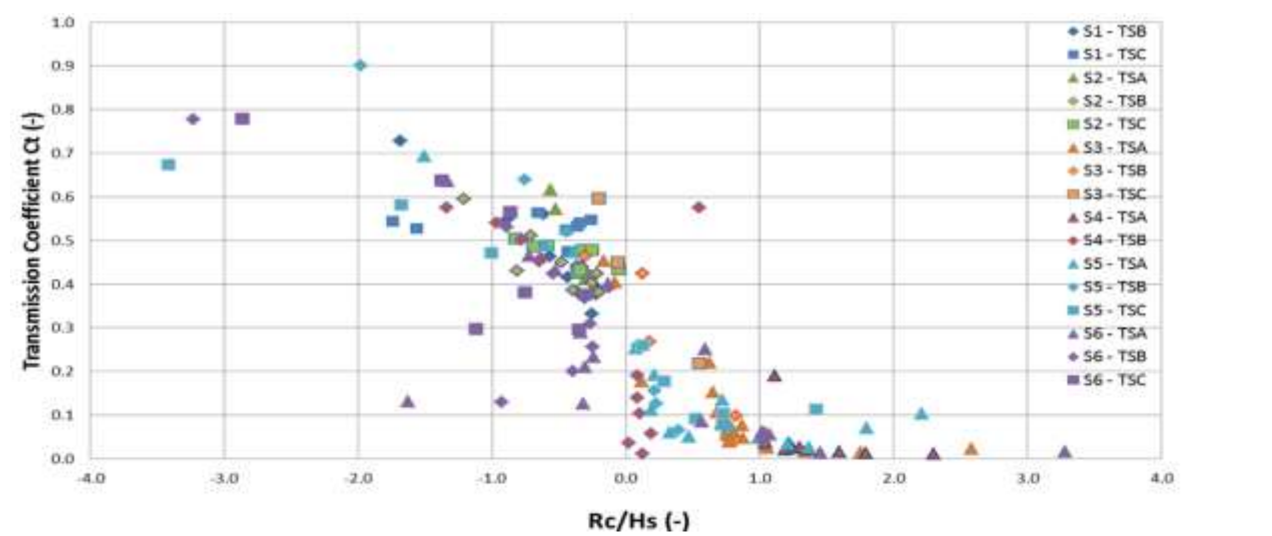



**Figure 7.41** **Wave transmission ($C_t$) against dimensionless freeboard ($R_c/H_{si}$)**

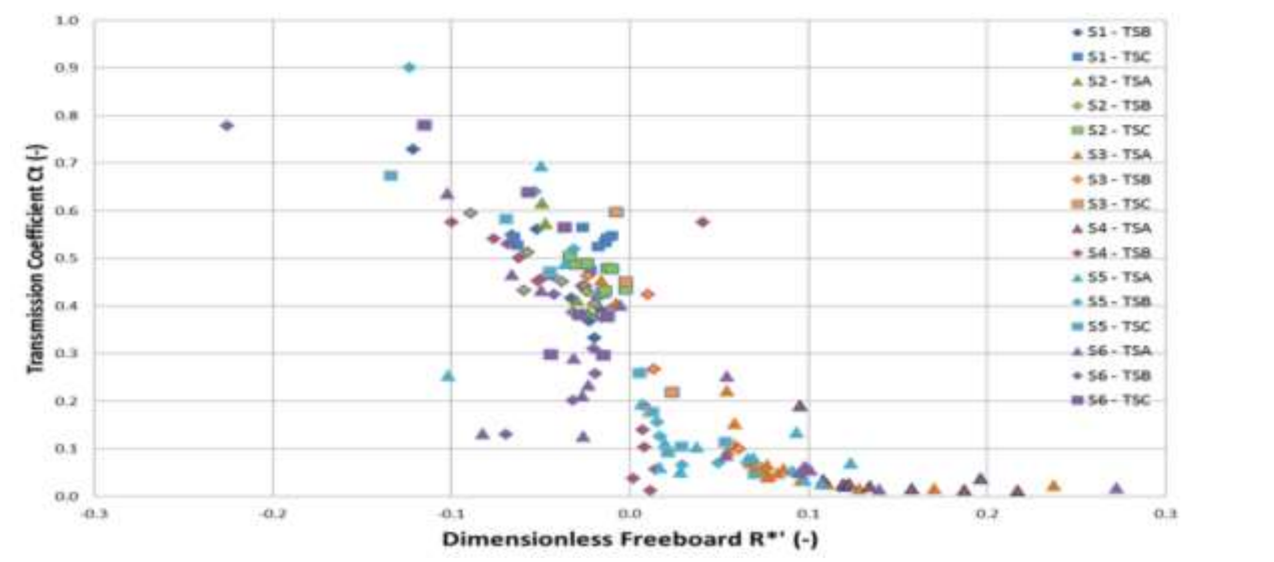


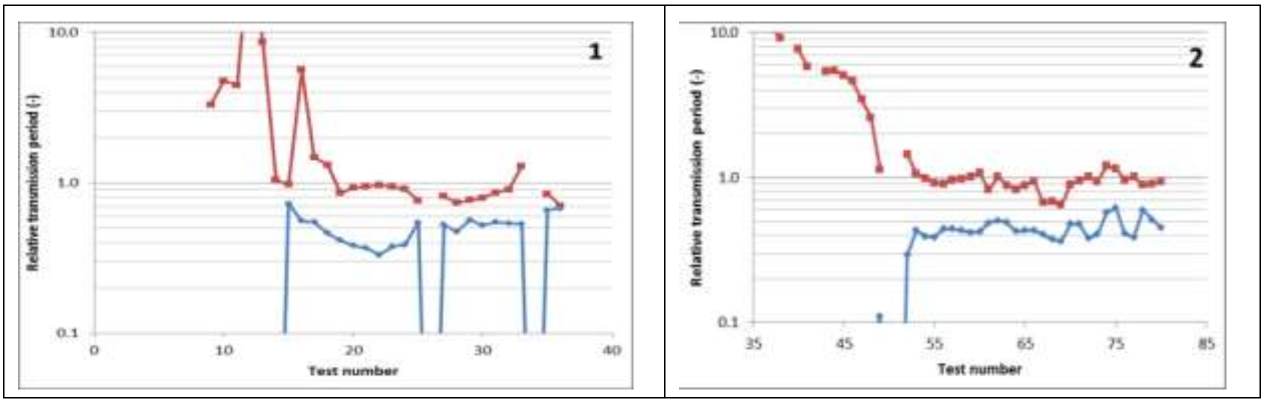

**Figure 7.42 Wave transmission ($C_t$) against dimensionless freeboard ($R_c/H_{si}$) /($\sqrt{s}(2\pi)$)**

**Effect of transmission on wave period**

Changes to transmitted wave heights as a function of wave height and crest level have been discussed above. An additional concern for wave overtopping studies in the lee of an extant or failed breakwater is whether the process of transmission will have significantly changed the dominant wave period. The analysis shown in Figure 7.43 has shown relative wave periods (simplified here to $T_t/T_i$) plotted as red points / lines. A logarithmic scale has been used to improve detail around $T_t/T_i = 1.0$. This does reveal that not all of the data are useful, especially when transmission given by $C_t$, is low. Values of $C_t$ have therefore been added in blue, noting that $C_t = 0$ do not plot on the logarithmic scale.

Ignoring values of $T_t/T_i$ for tests with very low levels of wave transmission (say $C_t < 0.3$), we see little departure from $T_t/T_i \approx 1.0$, with most relative periods falling in range $T_t/T_i \approx 0.8 – 1.5$.

For studies of wave overtopping in the lee of a collapsed breakwater, it may be appropriate to explore the influence of an increase of wave period up to $T_t/T_i = 1.5$, but these data do not justify any systematic change of transmitted period relative to incident.



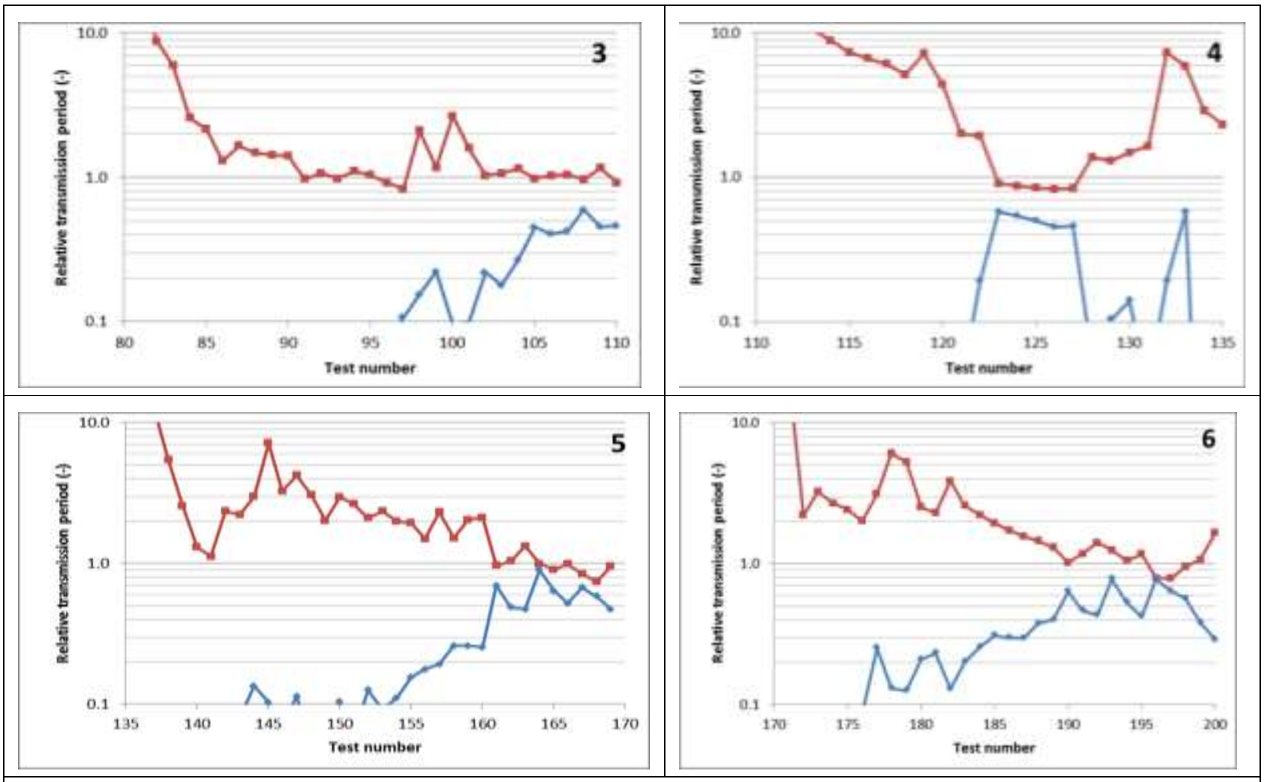



**Figure 7.43 Changes of transmitted wave period, Series 1-6**

## 7.6 Post collapse reshaping

Post-failure, the geometry of this (non-engineered) mound may be characterised by slope angles front and back, and the crest elevation. The most detailed results are given in the profile graphs of which a simple summary is shown in Figure 7.44 . It has been noted previously that the back slope is often much less regular, influenced strongly as it is by the form and rate of collapse of the rear wall. These rear slopes may not be representative of prototype walls in the longer term, especially where such failures involve material deterioration, and therefore reduction in size of the parent materials.

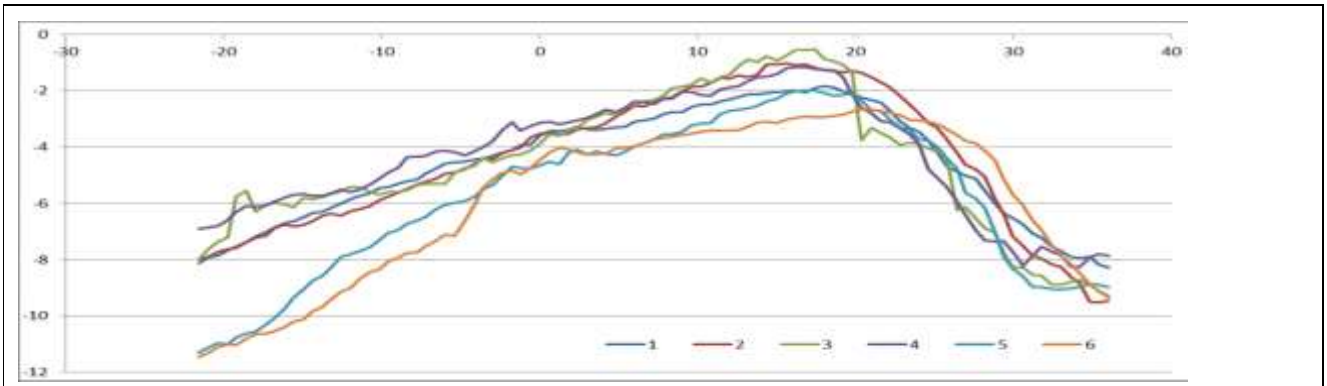



**Figure 7.44 Example post-collapse profiles, Series 1-6**

## 7.7 Processes of wall failure

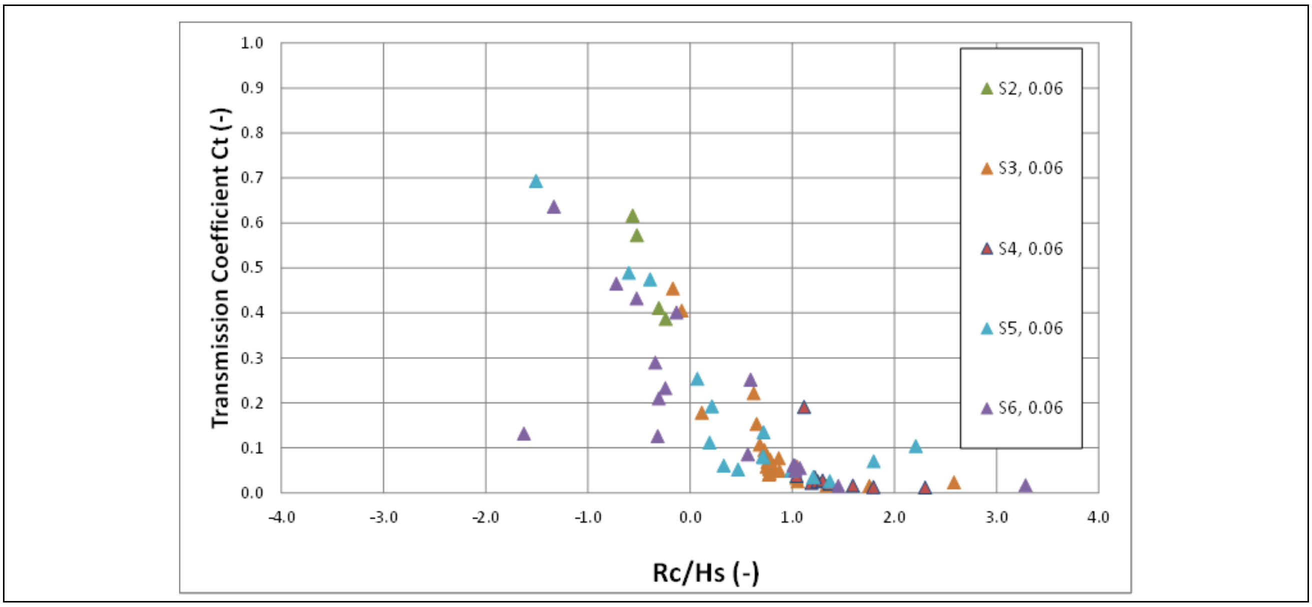
It is important to note that these were exploratory tests, intended principally to identify likely levels of wave transmission over failed blockwork breakwaters. In the study design, it was noted that the unit block size used to form the walls in these experiments probably exceeded block sizes in most historical walls. Up to the point of wall collapse however, the block size itself will not have significantly distorted the collapse process. The main distortion, delaying the process of wall failure, was the process of arching, initiated by (initially) unrepresentative friction between the blocks and wave flume walls. This frictional restraint in the earlier test series will probably have raised the effective wave height at which the wall started to fail.

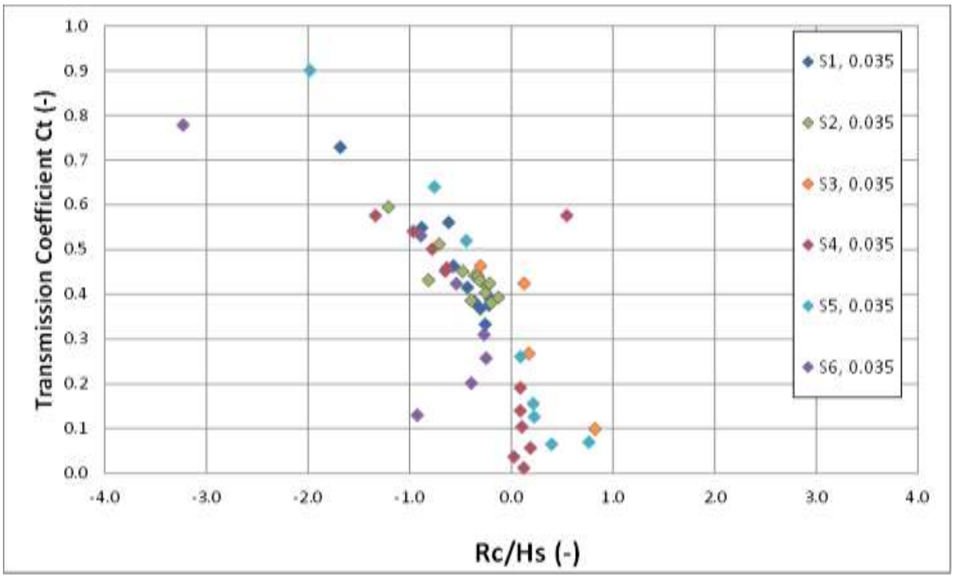
In considering the post-failure behaviour, it is noted that most early blockwork breakwaters used stone blocks of density ~ 2700kg/m$^3$, whereas these tests used concrete blocks of density equivalent to 2400kg/m$^3$. Whilst the model blocks were larger, so presented greater area to wave forces, they were also lighter, so were potentially less stable under wave attack.

## 7.8 Wave transmission over failed walls

The presentation of wave transmission measurements above links $C_r$ and $C_t$ to the damage process, but do not of themselves help predict wave transmission. Values $C_t$ were therefore plotted against dimensionless freeboard, $R_c/H_{si}$, in Figure 7.41 using mound crest levels from the profile measurements and the test water level to calculate $R_c$. The attempt in Figure 7.42 to explore whether an alternative approach by Powell & Allsop (1985) could reduce scatter by was judged not worth the increase of complexity in the plotting parameter.

Results for each of the wave steepnesses tested have been plotted separately in Figure 7.45. As presented, these data show that there were very few instances when the damaged mound fell below $R_c/H_s$ ~ -2, or where wave transmission exceeded $C_t$ ~ 0.6. The few instances of higher transmission arose from inherently smaller wave conditions. At first pass therefore, these two limits might safely be used in any initial assessment.





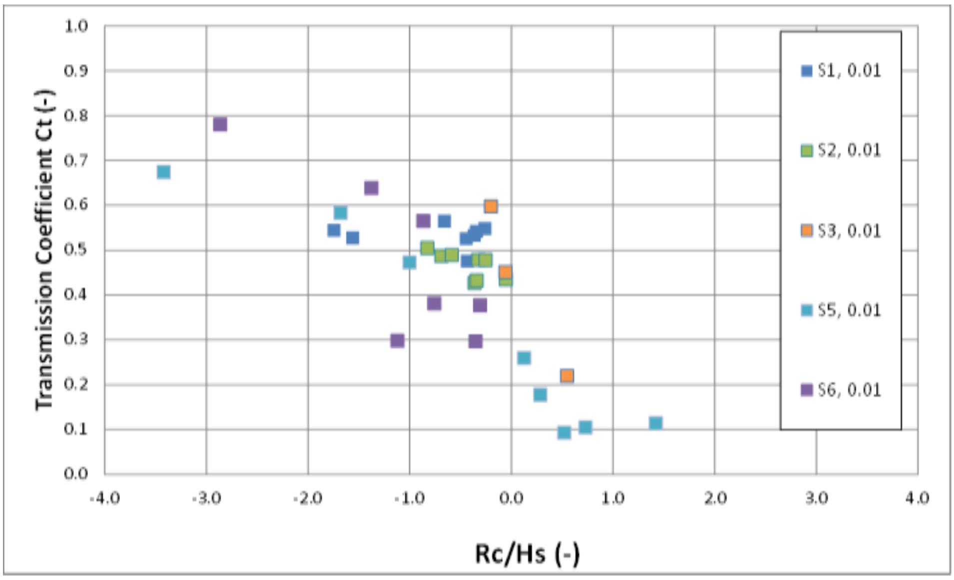




**Figure 7.45 Wave transmission, $C_t$ vs $R_c/H_{si}$: s= 0.06; 0.035; 0.01**

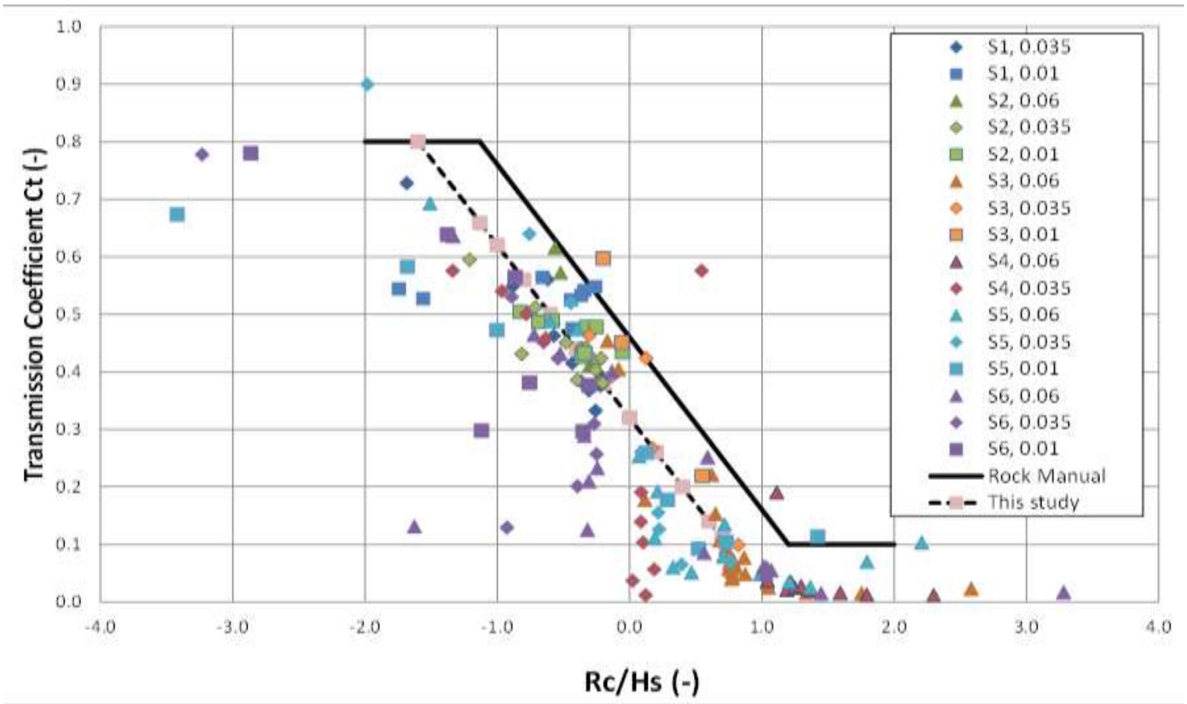



**Figure 7.46 Prediction graph for $C_t$ showing Rock Manual and new prediction lines**

The transmission results have therefore been combined in a single overall graph in Figure 7.46 to which has been added the (intentionally conservative) prediction line given in the Rock Manual. That prediction line is confirmed as pessimistic for these test results, so a revised set of (central) prediction lines have been fitted to the data from this study:

$C_t$=0.8 for $-4 < R_c/H_s < -1.6$

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

$C_t$ =0.1 for $0.7 < R_c/H_s < 3.0$

# 7 MODEL TESTS OF ‘ORPHAN BREAKWATERS’

# 8 LESSONS LEARNT

In analysing the 'lessons learnt' by breakwater engineers through the period of interest (1670-1910), it is useful to proceed sequentially where possible, although acknowledging that developments in understanding of hydrodynamics, availability of new materials and equipment, and of engineering techniques, did not follow linear sequences, perhaps more an iterative spiral with occasional reverses. The key problems are clarified in 8.1. Advances in understanding of hydrodynamics are identified in 8.2; improvements in construction in 8.3; and advances in knowledge management in 8.4.

## 8.1 The problems

The first breakwater reviewed here was at Tangier, discussed here in 2.3 and 13.2. Initially constructed as a classic 'blockwork wall on a rubble mound', the advancing construction was frequently damaged, and therefore took much longer and cost considerably more than budgeted. A potential solution to the construction difficulties was afforded by the innovative concept of using timber caissons. These appear to have been entirely successful from an engineering perspective, but the harbour was abandoned in 1683, and the 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 (see chapter 10), the lessons of the Tangier caissons were not learnt, and the experiment was not repeated in the UK possibly until the deployment of the Mulberry harbour caissons in the 1940s.

Jumping on from the 1680s to the decades after 1830, it was clear to civil engineers involved in design and construction of harbours in the UK that they had a number of problems, some manifesting themselves even during construction. The absence of settled design methods is highlighted by the multiple views, often disagreeing, taken during ICE discussions, see especially sections 11.1 and 11.2 summarising discussions to papers by Jones (1842) and Russell (1847). Yet the UK government had an urgent need to expand 'harbours of refuge', see 2.5 and Chapters 11 and 13.[23] Through the 1850s and 1860s, the construction of the new Admiralty breakwater at Alderney suffered repeated failures. Each stage of design revision showed only partial success (see sections 2.7, 6.3, and various parts of Chapter 11). Papers at *ICE Proceedings* were discussed by many engineers of the day, perhaps attracted by potential contracts for further harbours of refuge. These discussions certainly focussed on analysing the problems at Alderney, Dover, Holyhead. Some discussers also used the opportunity to promote other solutions (e.g. the discussions to Scott (1858) in section 11.7).

Elsewhere in the Channel Islands, the breakwater at St Catherine's on Jersey was completed in 1855 without incident, but the nearby breakwater at Archirondel was abandoned at a very early stage in 1849, and never proceeded further (see 2.8 and 13.14). Here the literature was generally silent until the book by Davis (1983).

In Scotland, Portpatrick harbour was abandoned by its mail contract by 1862, losing its major income, and hence *raison d'etre*. This was followed relatively quickly by the collapse of parts of both breakwaters (section 13.13). At Wick, Stevenson's new breakwater lost its outer end to storm damage in 1870 before construction was complete, followed by collapse of most of the breakwater trunk, see 2.11, 6.2, and 13.19. On a much smaller scale, the breakwater at Skateraw collapsed sometime between 1855 and 1890, and that at Greve de Lecq (Jersey) in 1879 and again in 1885.

At Dover, construction of the Admiralty Pier extension during the 1870s cost much more than expected, and the parapet wall was swept away in 1877, see 2.13 and 13.12.

The reasons for these failures were multiple, but the primary causes may be summarised:

[23] It may be useful to note that their successors in the 1970s and 1980s faced somewhat analogous problems in the multiple failures of rubble mound breakwaters armoured by concrete armour units, but by then mechanisms for communicating and analysing failures and discussing solutions were much more rapid and pervasive than a 150 years earlier.]

- Weaknesses in the prediction of wave heights.
- Little or no understanding of the effects on incident waves of natural shoals or foundation mounds, shoaling waves up, and inducing damaging breaking against the super-structures, coupled with the absence of wave load prediction methods, see particularly the analysis in sections 6.2 and 6.3).
- Difficulties in lowering the foundation level of the superstructure walls, leaving them in the 'danger zone' caused by the impulsive breaking above.
- Weaknesses of (stone) blockwork walls under wave loads, compounded by the complete absence of any method of analysis, see section 5.7.

Identifying these problems, and the responses of engineers of the time, have formed a substantial part of the present research. A synthesis of key findings follow, arranged under these general problem areas.

## 8.2 Improvements in prediction methods

### 8.2.1 Predicting wave conditions

In his 1874 handbook discussed in 11.8, Stevenson notes that he had presented in 1852 a simple rule for wave height (H in feet) given 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 giving "*much greater*" wave heights. In his manual of 1885, Vernon-Harcourt offers no significant improvement, and although Shield (1895) discusses more wave height examples, he still cites Stevenson's simple rule relating wave height to fetch length. Shield (1895) does however include a modified version for shorter fetches.

Many observers record extreme wave conditions where they could be determined from visual observations, but generally without coincident wind measurements. It may be concluded that throughout the period of this study, designers would never have had reliable wave predictions. They would instead have had to rely on estimates using Stevenson's or (later) Shield's simple formulae, then amplified by anecdotal wave height records such as those quoted by Shield in 11.11. Sadly, the great majority of engineers writing papers or attending discussions are silent on their derivation of wave conditions. A notable exception very much later is the discussion by Paxton (2009) on Stevenson's derivation of waves at Wick (see section 13.19)

### 8.2.2 Wave shoaling and breaking

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 sea-beds or artificial mounds, and thus the generation of impulsive breaking. These processes remained unclear to most engineers through most of this period, in part due to a lack of well-defined terms (particularly for stages of wave shoaling and breaking), and a lack of any empirical or analytical formulae to describe the processes.

There were occasional glimpses of clarity, although often lost behind misjudgements. Before discussing the design and/or construction of Blyth breakwater, Scott (1858) (see section 11.6) rehearses a number of 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 or Scott Russell, 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 …"*and *"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."*

In much of this, Scott has laid the foundations for clearer understanding of the processes of wave shoaling and breaking. Uptake of his explanation above may however have been inhibited by the general lack of approval of Scott's proposals for breakwater construction - see the extensive discussion in 11.6. Again, the lack of any agreed taxonomy, or of any analytical framework, did not encourage later contributors to take this improved understanding further.

Of the three principal 'manuals' by Stevenson (1874), Vernon-Harcourt (1885) and Shield (1895), only that by Shield is (slightly) clearer on the processes of waves shoaling and breaking over a shoal or foundation mound. Shield is generally clearer on wave processes noting that waves at Peterhead reached $H_{max}$ ~8m, $L \sim$ 150m, in 13-15m, then starting to break at the -10m contour (not unreasonable for $H_b/h \approx 0.78$). Citing Scott Russell's comments on depth-limiting wave heights, Stevenson derives $H=0.4d$ which might however be an optimistic version of $H_{sb} \geq 0.55d$, see the discussion on depth-limited breaking in section 5.2.2, illustrated by calculations of shoaling and breaking in sections 6.2.2–4 for Wick, and section 6.3.2 for Alderney.

### 8.2.3 Wave forces / pressures

Most current engineering design analysis starts by identifying representative loadings. In discussing wave loads, Russell (1874) notes however that "*…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…*". This illustrates why it was so important that descriptions on previous designs weree illustrated by the types of drawings shown in *e.g.* Vernon-Harcourt's manual in section 11.12, or in some of the discussions elsewhere in Chapters 11 and 13, so that comparative stability might be assessed.

In terms of wave load predictions, the designer in this period could use Stevenson's measurements (see Sections 11.5 and 11.8), or back-analysis from known failures, or successes. For the wave condition at Peterhead discussed above ($H_{max}$ ~8m, $L \sim$ 150m), 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[24]. The methods that Shield used for these calculations are not clear, but approximations may be given by assuming breaking wave velocities approaching the wave celerity, perhaps given by $c = gT/2\pi$. So 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. On their own, these numbers are reasonable, but again the absence of any clear explanation of the method of calculation significantly under-mines their potential value.

### 8.2.4 Blockwork stability

With the prevalence of uncemented blockwork in main and/or parapet walls in this period, it is particularly sad that nobody hazarded any method to analyse stability of the blocks forming these walls It was appreciated during that the stability of individual blocks in a wall depended on inter-block friction, see discussions by Stevenson and Shield in chapter 11. The discussion by Stevenson (1874) on friction between blocks has been particularly highlighted in 11.4. Despite the importance of wall stability, but perhaps not surprising given the lack of any prediction method for wave forces, there remained through this period, and to the present, no prediction method for blockwork stability. It has become clear from work reviewed in section 3.4 that stability of individual blocks depends critically on the continuity of blockwork 'pinching' forces across or up/down the wall face, interlock and jointing, presence of inter-block keys or joggles, and on (low) probabilities of

[24] A number of analysis methods for wave forces by Minikin, Sainflou, Ito, Goda, Cuomo and others discussed in Chapter 5 were, all developed substantially later than this period (1670-1910). Where analysing safety levels in the case studies in Chapter 6, slowly-varying wave pressures have been predicted by Ito or Goda methods. Such loads act over relatively large areas of wall driving the main responses of sliding and/or overturning. Impulsive pressures / forces are much higher in magnitude, but of much shorter durations and of limited spatial extent. Impulsive load durations are generally inversely proportional to load magnitudes. High levels of aeration extend load durations with concomitant reduction in load magnitude, see discussion in Chapter 5.

impulsive breaking. Even now, there are no formulae to predict the stability of individual blocks. Research by Muller and co-workers have identified magnitudes of pressures in cracks between blocks. Example movement analysis for individual (loose) blocks have been completed by Allsop & Bray (1994) and Marth *et al* (2004), but none of this work has yet been extended to allow local stability to be calculated generically, see discussion in section 5.8.

## 8.3 Improvements in construction

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

For instance, the development of concrete blocks to replace hewn stone blocks after 1850-60, thus accelerating construction, and substantially increasing block interlock, had to wait until cement production had been commercialised, mechanical concrete mixers were able to supply the volumes needed, and steam-powered cranes were available to lift the larger blocks[25].

In turn, the use of helmet-divers to allow foundation blocks to be laid at lower depths was substantially assisted when those blocks were uniform, shaped to increase load transfer, and could be lifted by steam cranes. Perhaps the main work of levelling the foundation would have required mechanical grabs, again lifted by steam cranes.

These developments would have proceeded independently at first, but later with some linkages, so it may be clearer to discuss each development in turn.

### 8.3.1 Use of timber

Timber has the significant advantages of availability, tensile strength, and ease of working. It was therefore used to form frames for rubble (see Figures 2.1 and 2.2 and section 3.1), and at Cherbourg (section 2.10) in the latterly notorious timber cones. In virtually all such uses, however, the timber was soon attacked by rot and ‘worm’ and was seldom adopted for permanent works once *e.g.* iron straps became available to carry tensile loads as in Figures 12.4 at Cockenzie or Figure 12.16 from Granton, completed in 1863. Timber was however used increasingly for temporary works, as was highlighted for Dover in section 2.13 and discussed further in section 3.2. Availability of tropical hardwoods and of steam-driven piling machines increased the more general use of timber piling and staging, see section 14.2.

### 8.3.2 Masonry walls

Given the lack of any analytical advances in the design of masonry wall as discussed above, it is difficult to identify any substantial improvements in blockwork walls until the rubble fill between the outer skins was cemented, or masonry blocks were replaced by concrete blocks, see 8.3.6 below.

In the period covered by this research (1670-1910), the great majority of masonry breakwaters used blocks in horizontal courses, despite the strong arguments by Stevenson to the inherent superiority of slice-work, sections 8.2.5 and 11.8, with which this author has considerable sympathy.

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. During the period of damage to the Alderney breakwater (1850-70), it was argued that a substantial parapet wall section would add further weight to increase stability of the wall. Contrarily it was known, but could not be quantified, that high parapet walls did attract

[25] Production of cement in volume was initially very localised, so therefore was its use, see section 8.3.6

substantial wave forces. 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, and ill-resolved.

### 8.3.3 Level of wall foundations

Through the discussions in *ICE Proceedings* revealed in Chapter 11, 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 (see Figure 2.1).

In discussion to Scott (1858) General Sir Harry Jones states that it is "*well known that the violence of the sea did not extend more than 15 ft below low water*". In contrast, Mr Murray at the same meeting referred to "*general acceptance of the 12 ft below LW*" rule. Mr Walker also refers to "... *a mass of pierre perdu reached within 12 ft of low water*." Mr Parkes also quoted the 12 ft rule, although allowed that 18 ft might be needed at some sites.

In practice, at Alderney the original design (by Walker, see section 2.6 in Chapter 2) started with a wall foundation at LWOST, then quickly dropped to 12 ft below, and then to 24 ft below for the 1864 roundhead. In their later handbooks, Vernon-Harcourt (1885) and Shield (1895) both prefer at least 20ft below LW.

The reasons for the importance of this dimension became clear on Alderney (see sections 2.6 and 6.3), and other breakwaters like Greve de Lecq (see example section 2.13.2). Firstly, as shown in the Wick and Alderney case studies in sections 6.2 and 6.3, a foundation mound may cause waves to shoal up and then break onto the wall. Secondly, a foundation mound that is itself loosened by wave action reduces foundation support for the blockwork above, allowing wave forces to move (and/or remove) individual blocks, leading to potentially rapid collapse of the wall. These latter processes are discussed in section 3.3 and illustrated in the physical model tests in Chapter 7. Any present-day decisions on the level of a mound foundation would now be guided by the need to avoid causing impulsive breaking, so should use analytical methods discussed in Chapter 5, see for instance Figure 5.5, and then applied in Chapter 6.

### 8.3.4 Use of caissons

One of the mysteries arising from the present research is that no British engineers appear to have followed the (apparently successful) use of caissons at Tangier, see sections 2.3 and 13.2. Their use would appear to have significantly speeded up construction and Routh (1912) seems not to have recorded any deleterious aspects. Yet construction with caissons was virtually never referred to again in the UK during the period of interest here. Perhaps the ill fate of the Cherbourg cones (see section 2.10) dissuaded later designers from using timber as a containment system for rubble, awaiting the development of thin-section reinforced concrete from which to build concrete caissons.

### 8.3.5 Slice blockwork stability

Stevenson (1874) argues very strongly the advantages of slice-work, particularly its capability in absorbing uneven foundation levels or support. He contends that this form of construction is substantially more stable than blockwork in horizontal courses, certainly in absorbing differential loadings and/or foundation support. Later versions using concrete blocks were cited by Bartholomew (1870) and Vernon-Harcourt (1885). In discussion to the paper to ICE by Buchan (1984) on Peterhead, Mike Leonard of Coode & Partners comments on the use of sloping or 'slice' blockwork, particularly to accommodate early settlement, see section 13.15. But as for many other 'gaps', the reasons for this omission are not recorded.

### 8.3.6 Concrete blocks replacing masonry

Ordinary Portland Cement (OPC) was patented by Aspdin in 1824, but it took some decades for production volumes to build until it was possible to source cement in volume for marine works. Use of concrete blocks becomes noticeable around 1855-60. At Rye, Winder (in discussion to Scott, 1858, see section 11.6) mentioned a pier formed by a single column of 10t concrete blocks. Concrete mainly replaced stone blocks and rubble, except on wearing faces. Here granite was still preferred, perhaps cast into concrete blocks during their manufacture to provide the dimensional continuity.

At Peterhead constructed around 1892, described by Buchan (1984), the main superstructure was formed by concrete blocks, jointed within courses by concrete joggles, section 13.15. Outer blocks included ashlar facing. Similarly, the outer harbour breakwaters at Dover were formed entirely by concrete blocks, apart from facing blocks into which were cast granite facing pieces / blocks, see sections 2.8, 6.4 and 13.12. The major advantages of concrete blocks were: considerably less work (time and cost) in dressing stone blocks; better interlock and jointing allowed load transfer transversely and vertically; individual concrete blocks could be substantially larger than stone blocks, say 40t vs 2t; elimination of granular fill between masonry walls.

### 8.3.7 Use of mass concrete

The use of mass concrete in forms is relatively seldom covered by papers or hand-books, so even though it was new to breakwater engineers in 1850, it was perhaps already regarded as routine after 1880, but a major difficulty in marine construction will have been in securing the formwork against wave forces.

At Aberdeen (1874-79), William Dyce Cay used fabric bags containing liquid concrete (5-16t per bag) to conform to the bedrock, and to be built up in layers. This approach allowed a stable foundation to the wall to be formed in stages, occupying little of the space of a mound, and avoiding the effect of a mound in ‘tripping’ incoming waves into breaking against the wall. This was deemed a success and later extended to 50t or 100t bags for the outer extension of the North Breakwater. Vernon-Harcourt (1885) shows a section through the placement hopper barge in Figure 3.15. Each bag occupied around 22.6m$^3$ when laid out within the specially built hopper barge, and received 50t of liquid concrete. Once the barge was winched into position, the bottom doors were opened allowing the bag to fall to the seabed. Multiple layers of bags were used, longitudinal for the first two courses; transverse for the upper courses. The same barge was used at Fraserburgh (1875-1882), discussed in section 12.6. A larger barge was used at Newhaven in 1884 to place 104t bags (47.1m$^3$), see Carey (1886) discussed in section 13.8.

At Peterhead, Sir John Coode (1888) designed the South Breakwater with a rubble mound up to -9mLW surmounted by a vertical wall formed by concrete blocks of 25-50t, see section 13.15. Working from the south side of Peterhead Bay, the first 108m were however formed by a wall of in-situ concrete topped by granite paving. Then the next 174m of the South Breakwater were founded directly onto rock, mostly using mass concrete in frames to raise to level, placed by bottom-opening bags or skips, occasionally supplemented by large hessian bags filled by concrete. The foundation initially at -9mLW was up to 4.4m thick, later (from 1897) lowered to -13mLW. The main superstructure was formed by concrete blocks, jointed within courses by concrete joggles. Outer face blocks included ashlar facing.

Kidd (1891) describes construction of a pier for the Skinningrove Iron Company[26] which produced hydraulic cement using blast-furnace slag. Somewhat unusually, Kidd formed a 2m high concrete foundation mound without formwork up to about 0.3m above MLWS placed by hopper boxes holding 0.6m$^3$ at a time (see Figure 14.5). The side slope was formed at about 1:1, and the top surface of the concrete was trodden down and

[26] North Yorkshire where ironstone workings in 1848 initiated industrialisation. Iron smelting began in 1874 and a jetty built in 1880 for seagoing vessels. Mining continued until 1958 and primary iron production until the 1970s

levelled by divers. This concrete foundation mound was placed in waves of up to 1-1.2m. Once in place for more than 12 hours, Kidd reports that little damage was caused by storms.[27]

### 8.3.8 Constructional plant

Above the ability to place much larger blocks, much more rapidly, the major advantage of the increased mechanization of marine construction was in the ability to place block low down in the water column. This reduced the mound volume, and hence its influence on wave breaking discussed previously.

For breakwater mounds, the simplest approach had been to deliver and deposit stone by boat, necessary in any case for any shore-detached breakwaters. For a breakwater of any size, this would require many vessels. Around 60 were used at Plymouth, see section 2.12. Examples of stone-dumping from rowed or sailed boats for Cherbourg were shown earlier in section 2.10.

The benefits of purpose-built staging from which to place blocks, rather than using the breakwater wall itself to provide the base for construction, were hotly debated in ICE discussions through this period, see Chapter 11. Both approaches had their advocates, although the use of timber staging (albeit temporary) grew in popularity as its advantages became clearer in reducing weather-dependency of construction operations, and new steam-powered cranes proved capable of handling longer timber piles and heavier concrete blocks. Earlier examples of timber staging are shown in sections 2.6 and 2.9 for Alderney and St Catherine's respectively, both constructed from 1847 onward, and more generally in section 14.2. In the lower tidal range at Alderney (but greater wave attack) an innovative adaptation was used to take the rail-wagon-delivered rock from the staging to the dumping barges without shooting the rock straight through the bottom of the barge! The reverse flow arrangement in sections 2.9 and 14.2 slowed the fall velocity enough to avoid sinking the receiving barge.

Beyond men or horses pulling rail wagons, major source of motive power before the availability of steam engines were treadmills or capstans and/or winches (block and tackle). Adapted from methods used commonly on sailing ships, these latter methods allowed man-power to lift and/or pull large weights. In 1839, it is recorded that 45 men used a capstan to winch ashore a 400t vessel at Ramsgate (Appendix A). Even by 1851, the crane-maker Stothert & Pitt exhibited a man-powered crane at the Great Exhibition (Appendix A).

### 8.3.9 Steam power, on land and on water

Whilst steam power had been used on static machines since 1781 (James Watt) and to drive other machines since 1801, it took more time before steam power was widely available, for locomotives, cranes, concrete mixing, or for vessels. In 1842, Clayton, Shuttleworth & Co started production of portable steam engines, producing some 2200 engines by 1856, see Appendix A.

By 1855 a steam locomotive was being used to move materials at the Mukkle Flugga lighthouse (Shetland), and in 1861 Stothert & Pitt exhibited a 6t steam-powered travelling crane at an exhibition in Paris. The increased use of steam power on breakwater construction is illustrated by Cay (1874) in section 13.5, and by Kidd (1899) in section 14.2.

In reviewing the advances of steam powered machines, Pitt (1893) noted that "*... vast development ... has taken place ... is largely due to the introduction of concrete and the consequent evolution of ... methods of construction which depend on its use ...*". Advances in plant (rapidly) allowed the designer to increase heavier blocks, and at greater reaches, so in turn designers demanded greater capacity. Pitt discusses two kinds of mixers: continuous and intermittent (later termed batch) machines. In batch mixers, he identifies those devised by Messent, Lee, Ridley, and Punchard. Pitt notes the need for external power sources observing that Sir John Coode had used a mobile steam crane to both lift materials and power the mixers.

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[27] The absence of any apparent repetition of this suggest that Kidd might have been boastful or lucky. Few modern engineers would place underwater concrete in this way.

With breakwater and quay walls now formed mainly of concrete blocks, the blocks are moved using Goliath or traveller cranes running on rails. In his presentation, Pitt (1893) showed three iterations of Goliath cranes for harbours at Karachi, Gisbourne, and Peterhead, each with a steam powered traveller or crab operating within the 17m span.

The largest machines are usually block-setting machines, often known as Titans (originally non-revolving); Hercules (revolving or radiating); Mammoths (two very large machines used at Tynemouth). Pitt suggested that the first Titan was built in 1869 for export to Manora (Karachi), laying blocks of 27t, and proved up to 40t. The first slewing Hercules was made by Stothert & Pitt for East London, South Africa in 1876, with a proof load of 30t. The Titan at Peterhead designed by W. Matthews was designed for a working load of 50t, proved to 62.5t, and a radius up to 30m.

Most of the machines discussed by Pitt seem to have been designed for a single site, although re-assembly and re-use at a second site is mentioned in passing. Many of the larger machines were in turn designed or specified by the breakwater engineers, or their staff. It is interesting to note that the frames of some significant overseas cranes were fabricated locally from timber rather than iron / steel, probably with the key winding and slewing components being shipped in from the UK. On at least one site, the timber of the temporary works was incorporated into parts of the works after the block-setting work had been completed.

In discussions to that paper, Charles Walker noted that Messent concrete mixers had been in use since about 1870, assisting the increased use of concrete from about 1850, and Sir Benjamin Baker recalled a Millroy excavator in use before 1868.

On water, in 1817, P.S."*Tug*", launched by Woods Brothers at Port Glasgow became the first steamboat to travel round the north of Scotland to the East Coast. In 1840, the Official Navy List shows a further 70 steam-powered vessels were added to the Royal Navy. Even so, a critical date for the adoption of steam-powered vessels is probably the trial in 1845 between HMS *Rattler* (propeller) and HMS *Alecto* (paddles) won by *Rattler,* see section 14.3. From then onwards, the availability of small propellor (sometimes termed screw-) driven vessels rapidly became un-remarkable, and attracts no attention in the literature.

### 8.3.10 Diving bells and helmet divers

Diving bells have been used in breakwater construction for many centuries, possibly as early as classical Greeks or Romans. In the context of this thesis, diving bells have certainly been used around the UK since the late 1600s, see section 4.2. The crucial improvement came when compressed air was pumped into the bell, equalising the water pressure. Smeaton used a diving bell of his own design at Ramsgate in 1788. By early 1800 use of diving bells to level foundations for later block-setting at moderate depths became increasingly common.

Whilst diving bells were particularly useful in preparing the natural seabed and/or levelling foundation materials, the major advance in improving construction quality, and construction speed, was the increasing use of helmet divers. This allowed wall foundations to be set substantially lower than previously, see section 8.3.3. Charles Deane (1796-1848) invented a Smoke Helmet in 1823, and in 1828, Deane and his brother converted the concept to a diving helmet. By 1836 the Deane brothers had already used the helmet in various salvage operations, and had produced the first diving manual. Augustus Siebe (1788-1872) improved and commercialised the helmet, leading to production from about 1837.

Together with use of block-setting cranes, divers permitted placement of foundation blocks lower in the water column, thus further away from disturbing forces from waves reflected from the wall. John Jackson in discussion to Vernon-Harcourt (1873) cites six divers being used at Alderney (mid 1850s) at any one time, and Cay (1874) illustrated use of a team of 32 divers at Aberdeen in the early 1870s in Figure 4.5 (see discussion in sections 4.2, and Figure 13.4 in Chapter 13).

In discussion to Scott (1858), Heinke noted that "*...recent improvements in diving-apparatus had ... enabled the diving bell to be superseded.*" At Dover, "*a diver could now place eight blocks* ...6-7t ... *in four hours, whereas a diving bell could only manage four or five"*. He understood that currents of 3mph (≈1.3m/s) or depths greater than 30ft (≈9m) gave difficulties to divers.

Even with such limitations, it is clear from the rapid and open-handed adoption of helmet divers that this advance allowed considerable improvements over use of diving bells, and was certainly greatly superior to trying to work down onto an elevated mounds .

## 8.4 Transfer of knowledge

This research has confirmed the considerable levels of information available to breakwater engineers during the latter part of the period of interest through publication of *Minutes of the Proceedings of the Institution of Civil Engineers*, known later simply as *ICE Proceedings*. This was then supplemented by the periodical *The Engineer* which sped up dissemination and uptake of technical information.

It is difficult to document the transfer of knowledge *per se*, but it has been possible to highlight some of the advances that have been discussed, and in many instances debated at length in *ICE Proceedings*, see Chapters 11, 13 and 14, enabled by the emerging knowledge transfer methods.

### ICE Proceedings

The Institution of Civil Engineers was founded in January 1818 as the world's first professional engineering body. The main publication was *Minutes of the Proceedings of the Institution of Civil Engineers*, later known simply as *Proceedings*, first appearing in 1836. Over the years this has divided into many journals, the first being between Part 1 covering construction and design; and Part 2 covering research. The journals later split along general and specialist topics, now including 24 titles.

The early *Proceedings* reported meetings of the Institution, particularly the presentation of papers, and the discussions that followed them. Some papers were relatively short. Thomas Cubitt presented a note in 1841 of only 20 lines on testing to failure a brick arch. Other papers in the same issue on bridges or sea defences were often of only one or more pages length. Later Stewart (1841) presented a two page paper on the state of construction of Plymouth Breakwater, to which Mr Rendel gave discussion of approximately 1/3 page.

By 1852 (strictly 1852-1853 as the 'ICE year' runs from November to end October), *Minutes of Proceedings* reached some 600+ pages. Of that, some 43 pages were devoted to members' obituaries (termed Memoirs). Elections of members were recorded some seven times through the year. Reports of the AGM, Council, and the Annual Report take some 17 pages and other administrative matters a further 37 pages. Main papers, with their discussions are given 109 pages, and technical communications 402 pages, so it is clear that from the start, ICE attached a very high priority to conveying advances in technical knowledge to its members.

When the general area of the paper was particularly topical, even if the paper itself was of only moderate quality, the discussions afterwards (including by correspondence) were often lively and extensive. Example discussions to papers by General Sir Harry Jones in 1842, by John Scott Russell in 1847, by William King-Noel in 1848, and by Michael Scott in 1858 are notable, especially that to Scott (1858) where the discussion ran to some 70 pages, see section 11.6.

Most harbours and their breakwaters were publicly funded, so there was considerable interest in costs and rates of progress of active projects, especially Alderney, see sections 2.6, 13.4, and 13.10. ICE members also worked around the world so brought back experience of construction from afar, notable examples being from India and Ceylon, Egypt, Venezuela, USA, Algeria, France, and the Netherlands. Not only did the discussers give considerable technical details, and costs, but they appeared to be happy to give their opinions, often forthrightly. Despite the discussions being edited by the secretary of the meeting (one assumes an ICE staff member), the tone of the meeting discussions can often be discerned from the record in the *Proceedings*, see example discussions in Chapter 11.

A further copy of *ICE Proceedings* for 1862-1863 again ran to 600+ pages with about 110 pages on main papers (on railways) and 430 pages on technical papers. Membership elections were again reported through the year, obituaries are included at the end of the yearly volume, and some of these have been used as source material for the Biographies in Chapter 15.

By 1874-1875 *Proceedings* (including the paper by Dyce Cay on Aberdeen discussed in section 13.5) now also included *Abstracts of Papers in Foreign Transactions and Periodicals*, summarising some 60+ different papers in 135 pages. These short summaries (often little more than a page) offered 'heads-up' on a remarkably wide range of topics of interest to civil engineers, albeit in that era with strong emphasis on railways and cement.

Viewed from an era where there is no longer any material level of discussion to papers, whether in *ICE Proceedings* or in other journals, the vigour and (apparent) candour in the study period of the discussion to papers in *ICE Proceedings* are remarkable, and would appear to have had significant (beneficial) influence on the transfer of new knowledge and techniques. Particular examples highlighted in the historic reviews in Chapters 11, 13 and 14 supply substantially more information to the debate, as well as recording engineers' judgements on how or why advances or modifications might or have been found to work. Of particular note were discussions to papers by General Sir Harry Jones in 1842, section 11.1; by the Earl of Lovelace in 1848, section 11.4; Scott on Blyth in 1858, section 11.6; and Stoney in 1874, section 11.9.

### The Engineer and Engineering

*The Engineer* magazine was founded in January 1856 as a weekly technical magazine for engineers by Edward Charles Healey, an entrepreneur and engineering enthusiast. Healey had financial interests in the railways and his friends included Robert Stevenson and Isambard Kingdom Brunel. *The Engineer* covered engineering including inventions and patents, originally also including prices of raw materials. Whilst there was a strong emphasis on railway and mechanical matters, *The Engineer* had a close relationship with the Institution of Civil Engineers, and with far more frequent publication than *ICE Proceedings* (see section 8.3.1), *The Engineer* covered topical civil engineering including ICE matters.

Many of the short articles first appearing in these periodicals then appeared as more complete versions in *ICE Proceedings* or in the handbooks, particularly by Stevenson (1874) and Shield (1895), see sections 11.8 and 11.11 respectively. More details on the publications *The Engineer* and *Engineering* are given in chapter 10.

### Current publication of improvements in practice

In contrast to the lively and informative discussions to papers in *ICE Proceedings* during the period reviewed, much of current international journal publishing has become academically self-serving with almost no input from engineering practice. The notable exceptions are some of the journals published by the Institution of Civil Engineers (ICE) and the American Society of Civil Engineers (ASCE), where some practitioner papers are still included. The practice of writing discussions to papers (in the same journal) has however almost completely disappeared, at least in part because the academic 'reward systems' favour stand-alone publications above the continuation of debate or technology transfer.

## 8.5 Lessons not learnt

The main improvements in understanding, materials, construction technology, and therefore in design and construction, have been summarised in sections 8.1 to 8.3. Also touched upon in passing were areas where lessons were not learnt in the period covered by this study.

The principal such 'gaps' were in the description of the key hydrodynamic processes, starting with wave generation, but principally the processes of wave transformation by shoaling, refraction, and breaking. These gaps were compounded by the lack of clear definitions, e.g. of significant wave height rather than maximum wave height, or mean wave period. Even in the latest of the handbooks compiled in this period, by Shield (1895), advice on wave generation was still mainly confined to Stevenson's simplistic relationship between fetch distance and wave height, supplemented by anecdotal observations of maximum wave heights at selected locations, see section 11.11.

In a follow-up paper, Shield (1899) presented some discussion on wave kinematics, but still failed to give any systematic guidance on processes or effects of wave transformations, see section 11.12. Even less advice is available on wave breaking limits, little of it capable of being quantified generically.

The next 'gap' is in predicting the stability of blockwork walls. Even now there are no methods to calculate forces on individual blocks. During the period of interest, there was a little guidance from Stevenson on wave forces, but nothing either systematic or capable of being used to calculate global stability. It is probable that the first such guidance would be that by Hiroi (1919) in Japan, or (in Europe) Sainflou in 1928.

A further omission in design methods was any method to quantify hydraulic performance, especially wave overtopping and/or wave transmission.

## 8.6 Summary

This project has reviewed the design and construction of breakwaters built by British engineers between 1670 and 1910, mainly of the classic 'blockwork wall on rubble mound' form. The research has analysed the factors that lead to early failures, or to survival, and then the progress of improvements in design and construction in the later part of the period studied. What did it reveal?

The main problem that led to these failures was of understanding the key wave / structure processes (hydrodynamics). Understanding these processes was made more difficult as storm waves are inherently rare (i.e. generally occurring only during severe storms), often of short duration (from a few seconds down to fractions of a second), and almost always impossible to see (underwater and/or masked by spray). These difficulties were increased by the absence of a common taxonomy, and of the analytical tools that we would now take for granted, especially wave prediction and transformation formulae and/or hydraulic modelling (wave flume tests).

At its simplest, these breakwaters tended to fail as the breakwater walls came apart (disaggregating) under wave impacts. In the worst cases, this even happened during construction or shortly after, as it certainly did at Alderney and Wick. In the case studies in Chapter 6, this research has shown that these failures were due to severe wave action, aggravated locally by waves shoaling over the foundation mound, and plunging onto the wall. The size and form of the mound aggravated the wave breaking, which in turn will have destabilised the mound material itself, loosening the wall foundation. The vernacular form of construction and materials for blockwork walls that worked well on land were no longer appropriate in the sea.

Whilst neither wave loads, nor the effects of shoals / mounds on breaking waves, could be quantified in this period, it was gradually appreciated that foundation mounds had generally been set too high. At Alderney, the level of the wall foundation was incrementally lowered as construction advanced through 1850-60, see sections 2.7 and 13.3-13.5.

The task of founding the lowest blocks in the wall on a firm base had however been very difficult, indeed probably impossible, until the availability of helmet divers who could ensure that the foundation was level, and that the blocks placed tightly. Those advances did however allow foundations to be taken much lower than previously, ultimately eliminating the deleterious effects of the high-mound on wave breaking and loads.

The other advances that helped improve robustness and speed up construction were the replacement of hewn stone blocks by pre-cast concrete blocks, often of much greater size than stone blocks, and the use of steam-powered cranes to lower those larger blocks into position, see especially sections 4.1 and 14.2. The effects of these improvements are illustrated by the success of the outer breakwaters at Dover reviewed in sections 2.13 and analysed in section 6.4.

A series of hydraulic model tests described in Chapter 7 studied the key failure mechanisms and collapse processes, and have clarified how much wave protection can still be provided when breakwaters of this type do collapse.

Underpinning many of the improvements developed during the period of study was the increasing role of the Proceedings of the Institution of Civil Engineers in disseminating new ideas, new materials, and improved methods of construction. An aspect of ICE Proceedings of considerable importance was the international reach given by the short summaries of foreign language papers on interesting advances. These drew examples from most European nations, and further afield, with particular favourites in this period being on cement performance and testing, and rails, and iron and steel.

Whilst some of the papers read to ICE may have included useful material, of far greater assistance to this research have been the many and varied contributions by engineers discussing those papers, or maters tangential to the paper. Four discussion sessions reviewed were of particular interest, by: General Sir Harry Jones in 1842 (section 11.1); John Scott Russell in 1847 (section 11.2); William King-Noel in 1848 (section 11.4); and Michael Scott in 1860 (sections 11.6 and 11.7). Extending these were the main handbooks or manuals of the period by: David Stevenson in 1874 (section 11.9), Leveson Vernon-Harcourt in 1885 (section 11.12); and by William Shield in 1895 (section 11.13).

Despite these, critical barriers to improved design were the absence of standardised engineering language (taxonomy), and later of analysis formulae. Whilst not directly driven by the founding of the Engineering Standards Committee (ESC) in 1901, later becoming the British Standards Institute (BSI). standardising approaches will have substantially assisted the development and application of new analysis methods, for instance the adoption by BSI's Maritime Code (BS6349) of Professor Goda's wave force method published in Japan in the 1980s.

# 9 CONCLUSIONS

## 9.1 Research aim, programme and dissemination

To summarise the key objectives of the research, this project was focussed on determining for the period of interest here (1663-1910):

1) How did breakwaters of this period fail?

2) What were the main causes of those failures?

3) What protection (wave shelter) was afforded by failed breakwaters?

4) What changed over this period to reduce failures, and thus improve resilience?

5) How were improvements promulgated and adopted?

Two informal 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 may remain?*

The choice of start and end dates for the research were book-ended by construction of the breakwater at Tangier (1663-1683) and of the outer harbour breakwaters at Dover (1897-1910). The former project was abandoned, indeed the breakwater was destroyed, after about 20 years. In contrast, the new breakwaters at Dover have proved to be highly successful, structurally, economically, and commercially.

To answer the research questions, this project has analysed the main technical literature of the period, primarily in papers and discussion in the ICE Proceedings and leading handbooks, to distil key lessons for breakwater design and construction. I have described in Chapter 2 the fate of selected breakwaters forming the Channel Islands Harbours of Refuge, and drawn that together in a paper to ICE Engineering History, Allsop (2020). Having analysed the design and construction of a wide range of breakwaters of the period, as described originally in literature of the time, the series of case study calculations in Chapter 6 have examined the stability or failure of breakwaters at Wick, Alderney, and Dover. The results of these case studies have been summarised in two papers to ICE Forensic Engineering, Allsop & Bruce (2020a, 2020b).

An addition to the planned research was the inclusion of the 'orphan breakwater' physical model testing reported in Chapter 7. These tests gave some insights into failure processes and progression correlating with examples from the historical review and site observations in Chapter 2. The results in Chapter 7 also provided specific guidance on the level of wave transmission over collapsed breakwater mounds. The results of these tests and analysis have been published in the proceedings of the ICE Breakwaters conference by Allsop, Pearson & Bruce (2017), and then in ICE Maritime Engineering by Allsop, Pearson & Bruce (2018),

## 9.2 Understanding hydrodynamics

This research has demonstrated that the major problem in this period was the lack of any analytical framework to describe wave processes at these structures. The lack of an agreed taxonomy to describe those wave transformation processes substantially inhibited the wider adoption of key insights into the effects on wave breaking and wave forces, *e.g.* those by Scott (1858) discussed in sections 11.6-11.7. This lack allowed many confused and confusing descriptions of wave processes to be promulgated. Even when some of the key transformation processes had been described correctly to modern eyes, the lack of any analytical framework meant that those descriptions even when essentially correct, were not adopted more widely. The net effect was that the processes by which waves transformed over shoals and/or foundation mounds before breaking impulsively against the breakwater wall only became qualitatively appreciated towards the later part of the 19th century.

Given those persistent weaknesses, and despite a fog of unclear descriptions, this research has shown that some engineers in this period were able to identify key influences on impulsive wave breaking and the failure of blockwork walls as discussed in section 8.2. General rules on the occurrence of impulsive breaking as wave transform over steep beaches and or rubble mounds were described by Stevenson (1874) and Shield (1895, 1899), but Scott (1858) came closest to describing the type of dangerous wave breaking on mounds that caused failures. Scott however failed to develop or use any unifying definitions of the wave processes, and so other authors of this era failed to embrace his advances, nor did they develop any analytical or empirical methods to assist them be adopted more widely.

## 9.3 Why did these breakwaters fail?

What were the primary failure mechanisms? For reasons of familiarity and (perceived) economy, the most frequent form of breakwater constructed in this period was a rubble mound, surmounted by a blockwork wall. The rubble mound was taken to a crest level as low as possible whilst allowing the blockwork wall to be founded securely on the mound. But before the availability of helmet divers allowing the wall foundation to be lowered, this junction was often set too high to avoid causing impulsive breaking against the wall.

The walls themselves were formed by hewn stone blocks on seaward and landward faces, with the space between filled by un-cemented rubble fill. They failed primarily by unravelling or disaggregating of the blockwork walls, often precipitated by even very small movement of the rubble mound foundation.

Once blockwork forming the seaward wall lost continuity, relatively local wave pressures could transmit into the wall, and then drive blocks back out. I had already identified contributory mechanisms in 1994, but that was then amplified by Muller and co-workers, see Muller *et al* (2002), Marth *et al* (2004, 2005) reviewed in section 3.4, which is here put into the context of the new research in Chapter 5. Once a relatively small area of blockwork has failed, general wall failure follows rapidly as is illustrated here in Chapter 7.

## 9.4 Key technological successes

Over the period studied here, engineers found that a key to improving wall stability was to found the wall toe as low as possible, eliminating the malign effects of a high mound in promoting impulsive breaking against the wall. Many of the key areas of improvement discussed here contributed to this objective.

An interim measure to reduce block movement in the superstructure was to cement up the loose fill between the superstructure walls, in practice possible only after the availability of Portland Cement around or later than 1850. Further improvements in wall stability were given by bedding the blockwork in cement mortar, and then by replacing masonry blocks and loose fill by concrete blocks throughout. All of these would have strengthened the wall itself, as was done for the outer sections at Alderney, but without strengthening (and/or lowering) the foundation, this gave only a delay to failure.

The processes of blockwork weakening, then block movement and/or removal were made much more likely where the approach slope or mound was at a sufficient level, and incident waves of sufficient height and wavelength, to shoal waves up over the slope and break them impulsively against the wall. The analysis of these process is illustrated in the case studies in Chapter 6 where analysis methods developed by the author and co-workers from 1990s onwards were based on output from the PROVERBS EU research project, summarised by Oumeraci *et al* (2001), and Allsop & Kortenhaus (2001). The case studies in Chapter 6 analysed failures or successes of breakwaters at Wick, Alderney and Dover reported in papers to ICE Forensic Engineering, Allsop & Bruce 2020a,b).

Whilst appropriate quantitative analysis methods were not available until 1928 at the earliest (Sainflou), and in the main not until after 1985 (*e.g*. Goda and other formulae for wave forces discussed in Chapter 5), the influence of mound level on impulsive breaking was qualitatively appreciated by some designers. As soon as it became practical to lower the level of the mound / wall junction, many designers made that change, assisted by the developing availability of helmet divers to place (or assist placement of) foundation blocks, and then by steam cranes to lower blocks, see examples discussed in section 14.2.

Again, this research has identified how those improvements were made substantially easier by replacing stone blocks and rubble fill by precast concrete blocks, themselves shaped to give inter-block and inter-layer bonding. Such blocks could also be substantially larger than dressed stone blocks, reducing the complexity and uncertainties of construction, and increasing its speed.

Alternative ways to form foundations for walls lower in the water column were to place mass concrete underwater in forms, or to place large fabric bags filled with 'liquid' concrete, as at Aberdeen and Newhaven, see section 3.8. Another technique to absorb variable bed levels, and/or differential foundation settlement, was to construct the superstructure using slice-work as advocated by Stevenson reviewed in 11.9, see examples in section 3.7. Originally using thin stone blocks on edge, specially shaped concrete blocks were cast with key-ways between blocks, but allowing 'columns' of blocks to slide downward where support was variable. While slice-work had its advocates, relatively few designers followed this path, and its use became rare.

The culmination of the advances to reduce previous failures is probably best illustrated by the outer breakwaters at Dover Harbour completed in 1910, section 2.13. These breakwater walls were composed entirely of concrete blocks keyed or joggled together, formed to full depth without any foundation mound, so avoided any risk of impulsive breaking. My analysis in Chapter 6 of the stability of these walls against global sliding or overturning has shown that they retain a high factor of safety even under severe combinations of wave and water level, and achieved despite the absence of wave and wave load prediction formulae at the time of their design, section 6.3.

## 9.5 New materials and construction methods

The main advances in materials derive from the development, and commercialisation of Portland Cement, patented in 1823, but only starting to be available commercially by the mid-1840s. The availability of such cement allowed the use of concrete, in mass, in bags, or as precast blocks from about 1850 (Chapter 8). Use of concrete substantially accelerated construction, once the plant was available to handle larger weights, and dramatically improved inherent stability / resilience of the completed constructions, see discussions in Chapters 6 and 8.

The major improvements in construction efficiency and in improved stability and resilience of the completed structures occasioned by the new materials above, were only possible once new equipment and techniques came available. Developments and use of new plant, diving bells and helmet divers, Titan or Goliath cranes, and use of temporary staging, have all been highlighted through the thesis (see Chapters 3 and 4, and sections 14.2 and 14.3 in Chapter 14) and are summarised in Chapter 8. In each case, their use allowed the wall foundation to be placed deeper, and masonry blocks and loose fill to be replaced by larger concrete blocks. In discussion to Scott, Heinke reports (section 11.7) that use of helmet divers in the work to extend Admiralty Pier at Dover before 1860 had resulted in twice as many blocks being placed than had been possible using a diving bell. In contrast, Stoney persisted with a diving bell at Dublin, even up to 1874, see section 11.10.

The two changes that increased structure resilience were the reduction of foundation mounds, reducing wave breaking onto the breakwater wall; and the replacement of masonry by purpose cast concrete blocks, improving block stability. Other advances contributed to those improvements, but also substantially assisted construction safety and speed. Construction safety is only rarely discussed in the literature of the time, and improvements will have been intermittent, but the rates of construction were hotly debated by ICE members in the discussions reviewed in Chapters 11 and 14.

## 9.6 Knowledge transfer

A major success over the period of interest was the sharing of knowledge from development, designs, construction, and operations. Major routes for new information after 1836 were the papers read to the Institution of Civil Engineers, and the discussions that followed those papers. In the early years, those discussions could continue over subsequent evenings. When the general area of the paper was particularly topical, the discussions afterwards were often lively and extensive. Discussions to papers by General Sir Harry Jones in 1842, by John Scott Russell in 1847, by William King-Noel in 1848, by Michael Scott in 1858, and Bindon Blood Stoney (1874) have been reviewed in Chapter 11. The discussion to the paper by Scott (1858) is particularly notable, especially as the discussion ran to some 70 pages, see section 11.6, and has revealed the degrees of understanding and confusion on near-structure wave behaviour. The discussions to Stoney in section 11.11 covered many different ways to construct, but emphasised that there was no universal solution.

Most harbours and breakwaters were publicly funded, so there was considerable interest in costs and progress of active projects, especially Alderney, see sections 2.6, 13.4, and 13.10. ICE members also brought back experience of construction from overseas, especially from India and Ceylon, Egypt, Venezuela, USA, Algeria, France, and the Netherlands. Not only did the discussers give considerable technical details, and costs of new or different breakwater types, but they happily give their opinions. The tone of the meeting discussions can often be discerned from the record in the *Proceedings*, see Chapter 11.

By 1875 *ICE Proceedings* now also included *Abstracts of Papers in Foreign Transactions and Periodicals*, summarising some 60+ different papers in 135 pages. These short summaries offered 'alerts' on a remarkably wide range of topics of interest to civil engineers.

Viewed from the present where there is no longer significant (if any) discussion to journal papers, the vigour and (apparent) candour of discussions reviewed in this research have been remarkable. The open exchange of views within an engineering community would appear to have had significant influence on transfer of new knowledge and techniques. Particular examples reviewed in Chapters 11, 13 and 14 supply substantially more information to the debate, as well as recording engineers' judgements on how or why advances or modifications might or have been found to work, see particularly discussions to papers by General Sir Harry Jones in 1842, section 11.1; by the Earl of Lovelace in 1848, section 11.4; Scott on Blyth in 1858, section 11.6; and Stoney in 1874, section 11.9.

In contrast, much of current international academic publishing has become academically focussed with little input from engineering practice. The notable exceptions are some journals published by the Institution of Civil Engineers (ICE) and the American Society of Civil Engineers (ASCE), where practitioner papers are still included. Even in those journals, however, the practice of including discussions to papers has however almost completely disappeared, perhaps because the academic 'reward systems' so strongly favours stand-alone publications above the continuation of debate or technology transfer. This is a sad loss as the comprehensive discussions in early *ICE Proceedings* reviewed here at length in Chapters 2, 11 and 13, provided much new information and guidance, often superior to the original papers.

## 9.7 Epilogue

Designers started the main period of interest knowing that wave loads could be problematic, but lacking the tools to understand how or why. They could not predict wave conditions with any reliability, nor could they predict how those waves would transform over the rubble mounds on which most breakwater walls were founded. They had no method to predict wave forces, or to determine the stability of blockwork walls. But some engineers did develop anecdotal approaches, so the main causes of impulsive wave loads were (at least partially) mitigated in time.

Lowering the junction between mound and the wall was made much easier by the increasing availability of helmet divers, the capabilities of steam-powered cranes, and vessels. The strength of blockwork walls was improved substantially by replacing random rubble fill by cemented fill, then the hewn stone blocks by cast concrete blocks with keyways or joggle joints to ensure lateral and vertical load transfer. The final step, indeed that taken at Dover, was to remove the foundation mound entirely, reducing substantially the probability of impulsive wave loads.

The postscript to the lack of wave force prediction methods is that since 1910, the profession took about 80 years to develop analytical methods to determine wave and wave loads reliably.

# PART 2 HISTORICAL REVIEW

# 10 INTRODUCTION TO THE HISTORICAL REVIEW

## 10.1 Outline

This short introduction to Part 2 of the thesis introduces the historical review being described in the following five chapters covering:

Chapter 11 - **General** literature, material intended to be of generic use;

Chapter 12 – **Geographic** – uses the ICE Civil Engineering Heritage project and other reviews to summarise the extent and types of breakwaters around the UK.

Chapter 13 – discusses fourteen **Specific Sites** in more detail, from which are chosen the case studies covered in Part 1.

Chapter 14 - identifies key **Technologies** that contributed to the successes of these structures.

Chapter 15 – identifies **Biographies** of some of the key participants in design, construction and/or the ICE discussions.[28]

To put the later work into some historical context, it may be helpful to identify a few key dates, based in part on the simplified chronology by Morgan (1984) which has then been combined with example events given by Straub (1952), see Table 10.1.

A substantially more complete timeline is presented in Appendix A covering the period 1625 to 1910 with entries categorised under three headings:

Places and activities;

People;

Technology

**Table 10.1 Simplified chronology from 1781**

| Dates | Activity |
|---|---|
| **1781** | James Watt patented a rotating steam engine |
| **1793** | France declared war on England |
| **1797** | Spithead and Nore mutinies |
| **1798** | French force under General Humbert landed in Co. Mayo. |
| **1799** | Income tax introduced |
| **1800** | Henry Maudslay invents screw-cutting lathe, presages production of standard bolts etc. |
| **1801** | UK union with Ireland<br>Steam engine used to drive machinery |
| **1802** | Peace with France |
| **1803** | War with France |

[28] This chapter makes no claim to be original research, nor is it comprehensive, simply to give the reader background to key participants.

| Dates | Activity |
|---|---|
| **1805** | Battle of Trafalgar, Nelson defeats combined French / Spanish fleets |
| **1815** | Napoleon escaped from Elba, February; defeated at Battle of Waterloo |
| **1819** | Peterloo massacre at reform meeting kills 11, wounds 400 |
| **1820** | George III dies, George IV accedes |
| **1821** | Napoleon dies on St Helena |
| **1824** | Patent for Portland Cement by Joseph Aspdin |
| **1825** | Stockdon and Darlington railway opens |
| **1829** | Stevenson’s ‘Rocket’ wins race at Rainhill |
| **1830** | George IV dies, William IV accedes; L’pool & Manchester r’way opens |
| **1831** | Rural riots against mechanisation |
| **1832** | Great Reform Bill |
| **1833** | Factory Act limits child labour |
| **1834** | Slavery abolished in British Empire |
| **1837** | William IV dies, Victoria accedes |
| **1839** | Chartist riots |
| **1840** | Penny Post started |
| **1844-45** | Railway boom, 5000 miles of track added |
| **1846** | Corn Law (Importation Act) abolished |
| **1847** | Construction of Alderney breakwater started |
| **1850** | Compressed air used for foundation work for Medway Bridge |
| **1851** | Great Exhibition |
| **1854-56** | Crimean War |
| **1857-58** | Second Opium War in China |
| **1858** | Indian Mutiny |
| **1859** | Publication of Darwin's "Origins of Species" |
| **1860** | Prince Albert dies |
| **1862** | Limited Liability Act |
| **1863** | Construction of Stevenson’s breakwater at Wick started |
| **1867** | Monier’s first patent for reinforced concrete |
| **1869** | Suez Canal opened |
| **1875** | Disraeli buys controlling interest in Suez Canal |
| **1880-81** | First Boer War |

| Dates | Activity |
|---|---|
| **1882** | Britain occupies Egypt |
| **1885** | Burma annexed |
| **1887** | British East Africa Company chartered |
| **1889** | British South Africa Company chartered |
| **1897** | Construction of Dover outer harbour started |
| **1898** | Sudan conquered |
| **1898** | German naval expansion |
| **1899** | Second Boer War (to 1902) |
| **1901** | Victoria dies, Edward VII accedes |
| **1904** | Anglo-French entente |

## 10.2 History of Civil Engineering by Straub (1952)

This history of civil engineering was originally written in German, later translated into English. It does not purport to specialise in coastal or harbour engineering, let alone breakwaters, but does identify a few relevant events / activities bearing on the generality of civil engineering construction.

On ancient harbours, Straub contrasts Greek and Roman harbour building, noting Roman preparedness to select sites '*where everything had to be created by human effort*'. He describes briefly the ports at Alexandria, Civitavecchia and Ostia, noting that the '*substructure of breakwaters and jetties consisted normally of stone ballast*', but also '*foundations consisting of rectangular blocks ... which weighed up to 9 tons, and were laid out in the shape of regular walls*'. He notes use of underwater concrete of broken stone, lime and pozzolana. Vitruvius describes a method for the construction of jetties consisting of artificial blocks of pozzolana concrete.

**Table 10.2 Summary of 'Great Engineers' (after Straub, 1952)**

| Dates | Engineer and Activity |
|---|---|
| **1663-1729** | Newcomen |
| **1711-1763** | Abraham Darby II (smelting of iron, 1735) |
| **1716-1772** | James Brindley (canal builder, worked with Smeaton) |
| **1724-1792** | John Smeaton (lighthouses and steam engines) |
| **1736-1819** | James Watt (steam engines) |
| **1750-1791** | Abraham Darby III (smelting of iron, 1735) |
| **1756-1836** | John Loudon Macadam (road construction) |
| **1757-1834** | Thomas Telford |
| **1761-1821** | John Rennie (the Elder) |
| **1771-1833** | Richard Trevithick |
| **1779-1855** | Joseph Aspdin |
| **1781-1848** | George Stevenson |

| | |
|---|---|
| **1788-1829** | Tredgold |
| **1789-1857** | Cauchy (initially harbour works at Cherbourg) |
| **1803-1859** | Robert Stevenson |

On civil engineering machinery, e.g. hoists / cranes, construction machines, diving bells etc., Straub notes that power to work cranes and hoisting gear was for many eras supplied by treadwheels as "*known to the Romans*". Straub refers to hydraulic machines, particularly bucket elevators and water screws for drainage. In bridge construction in Paris, Perronet used a water wheel driven by the river for the lifting of a pile-driver ram (c1740?). Belidor's 'Architecture hydraulique' shows a drawing of a dipper dredger used at Toulon operated by cranks and it identifies "*enclosed boxes with drop bottom used for the pouring of underwater concrete"* possibly still in the mid 1700s. In 1774, Groignard built a large dry dock in Toulon, a wooden floating caisson of 31m x 100m base and 11m height, partitioned into 8 compartments (reported by P Bonato (1882) Annali degli Ingenegnare ed Architetti Italiani.). Coulomb (1736-1806) proposed a wooden caisson with chamber for underwater working – a diving bell,

Late 1700s – development of flanged wheels and cast iron rails to replace oak. Cast iron rails replaced by malleable iron during 1810-1820s. First steam-powered road locomotive ran on a Welsh colliery tramroad in 1804 hauling 13 tons at 5mph, soon increased to 40 tons at 5.5mph at Killingworth colliery. Opening of the Liverpool – Manchester railway line in September 1830 confirmed steam locomotives over stationary steam engines.

Joseph Aspdin (1779-1855), mason and building contractor patented the manufacture of 'artificial stone', a 'cement of artificial stone' called Portland Cement, in October, 1824. Aspdin was assisted in the development of cement chemistry and production by J.C. Johnson (1811-1911), manager of a cement factory during 1840s.

Availability of steam motive power, and of wrought or malleable iron machines allowed development of machines for: concrete / mortar mixing; drilling; cutting; hammering; lifting; and transporting. Main advances in mechanical engineering in Britain drove improvements in construction. John Rennie, the Elder (1761-1821) used steam engines to operate pile driving and pumping in London Docks in 1801. Robert Stevenson (1803-1859) first used a steam hammer for pile driving on the Tyne in 1846. The first mechanical stone-crusher was developed in America in 1858. Large mechanised concrete mixers were available in Europe from around 1850.

## 10.3 UK waterfront walls (Bray & Tatham, 1992)

The main interest of this compilation is UK waterfront or quay walls, although a few breakwaters and seawalls are included in passing. Early 19th century UK ports required relatively few breakwaters. Analysis of wall stability is biased almost exclusively to retaining wall geotechnics, with no hydraulic loadings.

**Table 10.3 Summary of 'UK Breakwaters' (after Bray & Tatham, 1992)**

| Breakwater and Dates | Reported by |
|---|---|
| Eyemouth North Pier (1767), | Skempton |
| Donaghadee (1821), | Rennie |
| Old Pier, Wick (1823), | Stevenson |
| Whitehaven west pier (1831), | Williams |
| Hynish, Argyll (1843), | Stevenson |

| | |
|---|---|
| Kilrush Pier, Shannon (1843), | Shield |
| North Pier, Hartlepool (1847-58) | |
| Alderney (1851-1864), | King & Bishop |
| Nether Buckie (1855), | Stevenson |
| Tyne North Pier (1855-1895), | Stevenson, Messant |
| St Catherine's, Jersey (1856), | Vernon-Harcourt |
| Dover (1866), | Vernon-Harcourt |
| Aberdeen South Breakwater (1873), | Cay |
| Aberdeen North Pier (1877), | Vernon-Harcourt |
| Holyhead (1876), | Hayter |
| Fraserburgh (1877), | Willet |
| Newhaven (1880), | Carey |
| Ardrossan (1892), | Robertson |
| Dover (1898-1909), | Bryson Cunningham, Wilson |

## 10.4 Institution of Civil Engineers and ICE Proceedings

Many learned societies were founded in the late 18th century and early 19th century. Groups of civil engineers had been meeting for some years, for example the Society of Civil Engineers formed in 1771 by John Smeaton, renamed the Smeatonian Society after his death in 1792. Formal engineering in Britain had been limited to the military engineers of the Corps of Royal Engineers. To provide a focus for the fledgling 'civilian engineers', the Institution of Civil Engineers was founded in January 1818 as the world's first professional engineering body.

Growth of the Institution was initially slow until the appointment of Thomas Telford in 1820 as the first ICE President. Telford commanded great respect within the profession and had already developed numerous contacts across industry and government. Telford was instrumental in increasing membership and getting a Royal Charter for ICE in 1828. this official recognition helped establish ICE as the pre-eminent organisation for engineers. It was only in 1847 that the Institution of Mechanical Engineers was established, (with George Stevenson as its first President).

The ICE moved into Great George Street in Westminster in 1839 having already started publishing learned papers, and discussions to those papers since 1836.

The main publication was *Minutes of the Proceedings of the Institution of Civil Engineers*, later known simply as *ICE Proceedings*. Over the years this has divided into many journals, the first division being between Part 1 covering construction and design; and Part 2 covering research. The journals later split along topics, now comprising in excess of 24 titles.

The early *Proceedings* reported meetings of the ICE, particularly the presentation of papers, and the discussions that followed them. Some papers were relatively short. For instance, Thomas Cubitt presented a note in the 1841 *Proceedings* of only 20 lines on the testing to failure of a brick arch. Other papers in the same issue on bridges or sea defences were often of only one or more pages length. Later Stewart (1841) presented a short (~2 pages) paper on the state of construction of Plymouth Breakwater, to which Mr Rendel gave discussion of approximately 1/3 page.

But when the paper was particularly topical, even when itself was of only moderate quality, the discussions afterwards, and by correspondence, were often lively and extensive. Example discussions to papers by General Sir Harry Jones in 1842, by John Scott Russell in 1847, by William King-Noel in 1848, and by Michael Scott in 1858 are notable, especially that to Scott (1858) running to some 70 pages, see Chapter 11.

Most harbours and breakwaters were publicly funded, so there was considerable interest in the costs and rates of progress. ICE members also worked around the world so brought back experience of construction from afar, notable examples being in India and Ceylon, Egypt, Venezuela, USA, Algeria, France, and the Netherlands. Not only did the discussers give considerable technical details, and costs, but they were happy to give their opinions, often forthrightly. Despite the discussions being edited by the secretary of the meeting (one assumes an ICE employee), the tone of the meeting can be discerned from the record in the *Proceedings*.

Further examples of the international reach of the ICE are given by short summaries of foreign language papers on interesting advances. These may have appeared annually, drawing examples from most European nations with particular favourites in this period being on cement performance and testing, and on iron and steel.

## 10.5 The Engineer, and Engineering

*The Engineer* was founded in January 1856 as a (generally weekly) technical magazine for engineers by Edward Charles Healey, an entrepreneur and engineering enthusiast. Healey had financial interests in the railways and his friends included Robert Stevenson and Isambard Kingdom Brunel. *The Engineer* covered engineering including inventions and patents, originally also including prices of raw materials. Whilst there was a strong emphasis on railway and mechanical matters, *The Engineer* had a close relationship with the Institution of Civil Engineers, and with far more frequent publication than *ICE Proceedings*, *The Engineer* covered topical civil engineering including ICE matters. An example edition in December 1878 reviewed new locomotive engines, and milling machinery from the 1878 Paris Exhibition; news of Aveling & Porter's reduction of workers wages; news on Edison's microphone and telephones; an Editorial on the constitution of ICE; Fisken & Co's 8HP traction engine; two legal cases; and an article on practical uses of electricity for lighting (by William Siemens). That issue concludes with an article on a steam barge winch.

The final print edition was in July 2012, but *The Engineer* magazine is now available online at: https://www.theengineer.co.uk/news/.

Separate was *Engineering*, a weekly journal founded in 1866 by Zerah Colburn (an American locomotive engineer), as a weekly rival to *The Engineer*. *Engineering* was largely funded by Henry Bessemer the English engineer and inventor known chiefly in connection with the Bessemer process for the manufacture of steel.

Colburn was a teenage prodigy with little formal education. Barely in his teens, he found work in Lowell, Massachusetts as an apprentice where America's first steam locomotives were designed. Colburn began a regular news-sheet – *Monthly Mechanical Tracts*. His first book, The Throttle Lever, became the standard U.S. textbook on building locomotives, and earned him respect from locomotive builders and train operators across America.

In 1853 he joined the *American Railroad Journal*, but parted after a dispute with the editor and launched his own weekly paper, the *Railroad Advocate*, later relaunched as *American Engineer*, a weekly paper reporting technical and business aspects of locomotive manufacture and railroad operation in America. It closed in 1857.

Colburn visited Britain to report on the (successful) state of Europe's railways for the presidents of America's railroads. The report was a success, but by 1858 Colburn returned to England to take up a job as editor of *The Engineer*. Here, Colburn joined both ICE and IMechE, giving frequent lectures and contributing to meetings. Colburn probably met Isambard Kingdom Brunel, and in 1860, he returned to America on the maiden voyage of Brunel's steamship, the Great Eastern. Colburn returned to England to take up his previous position at *The*

*Engineer* in London, but was dismissed four years later. In 1864 he was awarded a Telford Medal by the Institution of Civil Engineers for his paper 'On American Iron Bridges', and in 1869 he received a second Telford Medal for 'On American Locomotives and Rolling Stock'. He founded *Engineering* as a weekly rival to *The Engineer*. At *Engineering*, Colburn’s writing style and wide engineering knowledge gave the journal considerable success, overtaking *The Engineer* in circulation. Colburn committed suicide in USA in 1870 at the age of 38.

# 10 INTRODUCTION TO THE HISTORICAL REVIEW

# 11 GENERAL HISTORICAL REVIEW

This Chapter discusses key documents from between 1842 and 1899 that may be regarded as of ‘general’ relevance, that is not at a single site. These include manuals or guidebooks by Stevenson, Vernon-Harcourt and Shield; and ICE papers and their discussions that have more general application. Where possible, I have presented these discussions in sequence by the date of the original paper / presentation to ICE

## 11.1 Jones (1842) on breakwater sections, with discussion

[Biographical note: Lt-General Sir Harry Jones was commissioned as 2nd lieutenant into the Royal Engineers in 1808, being employed on fortifications at Dover. He joined Wellington's army through the Iberian Peninsula campaign of 1812–13, and at San Sebastian he was wounded, and captured. In 1815 he re-joined Wellington's army in occupying France. On return to UK, he was stationed at Plymouth, then in 1823 to Jersey, and in 1824 at the Royal Engineer Establishment at Chatham. In 1826 he was sent to Malta, north Africa, and Constantinople to report on their defences. In 1835 he was appointed to the navigation commission on the River Shannon, and in 1836 to the Irish railway commission.

In 1837 Jones was employed on special service to the Admiralty, later being appointed commanding royal engineer at Jersey. In 1845 he was appointed chairman of public works in Ireland. On the declaration of war with Russia, he was appointed Brigadier-General. Jones commanded British engineers, marines, and naval artillery ashore. In 1854 he replaced Sir John Burgoyne as commanding royal engineer. Jones was severely wounded at Sevastopol. In 1859 Palmerston appointed Jones as chair of the royal commission on defences of the United Kingdom whose report recommended a massive and expensive programme of fortification of naval bases, most of which were eventually built.]

In this 2-page paper, Jones objects to rubble slopes and advocates vertical breakwater walls formed by close-fitting hewn stones. The paper may have been presented as part of the greater debate on ‘harbours of refuge’ specifically to promote discussion at ICE, in which he was successful.

Lieut.-Colonel Harry Jones had made "*... several years' observation of the effect of storms upon the sea faces of breakwaters and piers ...*" from which this short paper was compiled. He started with rubble mounds, then often known as "*pierre perdu*" which followed construction of the cones at Cherbourg, "*...since then the general method of constructing sea-defences has been to throw down masses of stone, allowing them to form their own angle, subject to the effect of the sea ...*". He notes that "*... in many instances this rough foundation has been paved down to below low-water mark with squared blocks of stone ... and above this a wall is built of solid masonry ...*"[29].

Jones then contends that the approach of matching the slope of the breakwater to that of a natural shore was an error: "*no pier or breakwater constructed with a sea slope has been found to resist the effects of storms without considerable repairs and expense being subsequently required.*" In this, he would appear not to be accounting for the effects of reflections from the wall, nor of the mound on the form of wave at the wall.

He cites damage or failures at Kingstown Eastern Pier (200,000t of stone needed on the foreshore), Plymouth (more stone needed), Dunmore (iron chains fixed to walls to secure them!), Howth (3:1 slope insufficient – does he mean 1V:3H?), Ardglass (pier head and lighthouse washed away). At this point he is less than clear as to his concern on the inclination (batter) of the wall, or of the rubble mound. In contrast he cites piers at Old Dunleary and Kilrush.

Jones proposes that vertical ("*perpendicular*") walls should be founded on rubble mounds ("*pierre perdu*") from "*a little below the level of low-water spring-tides*", claiming that the French had partially adopted this

[29] I assume that here he is referring obliquely to the construction of Plymouth Breakwater, started in 1812, and perhaps only just completed in 1841.

approach for new works at Cherbourg. He noted that the section at Plymouth had been altered three times, extending the base and enlarging the sectional area. At Howth, "*damage is so extensive that the sea threatens to make a clear breach*". At Kingstown Pier he notes slopes of 4:1 and 5:1[30]. At Kilrush " *a sea wall with very slight inclination … sustained no injury …*". His recommendation was therefore for stones of "average size, square-jointed and well-laid, even without cement, forming an almost vertical wall … springing from a point as much below low-water mark as could be conveniently attained …". He notes that the French used walls of "*ashlar, filling them in solid with "beton", and then caulking all the ashlar joints with oakum*". He asserted that "*this kind of work was very durable"*.

In answering a discussion, Jones agreed that the greatest damage was due to "*the receding wave, particularly if the joints … were not well closed*".

In discussion, **Mr Rennie** felt that engineers were correct to use as a guide "*the natural inclination of the sea-shore*", but noted that "*the force of the waves acted more prejudicially upon the point above low-water than below it*". He could not agree with the general form proposed by Colonel Jones. He noted that an upper slope could adjust given the circumstances (geometry and materials) and "*the degree of violence of the action of the waves*".

**Mr Vignoles** partially agreed with Jones if meaning that the wall should only be commenced "*at such a depth below low-water as should prevent the violent action of the waves upon it.*" At Ardglass where the foundation blocks were of large ashlar, placed from a diving bell, the lower part remained intact even when the upper part was destroyed.

**Mr Gordon** had seen the rubble (*pierre perdu*) foundation at Madras taken to 3m below LW where it sat undisturbed at 1:1 despite the violent surf.

The President (James Walker) noted that stone blocks of 6-8t had been washed from the seaward face of Plymouth Breakwater (started in 1812), but that recent work with closely fitted square stones have been placed "*with the utmost care, and little comparative injury had been done since that methods has been adopted.*" He felt that for a structure exposed to heavy seas, "*a long slope for the sea face was essential*".

**Mr HR Palmer** did not believe that the forms suggested by the Author were well supported by observed facts. He drew an analogy with the shingle beach at Folkestone that sat at 1:9 under heavy seas. He further noted that some obliquity could be very beneficial. He had set his pier at Mount's Bay, Penzance, at 5° to the incident waves to reduce the forces. He attributed the failures cited "*by Colonel Jones more to defective workmanship than to faults in the principle of the structure*".

**General Pasley** noted that beaches of shingle or rock adjusted their slope to the waves, but had little effect on bluffs of well-cemented rock. He felt that for a perpendicular (vertical) "*wall constructed of large ashlar work well cemented … prejudicial action … would be avoided.*"

**Mr Bull** drew a parallel with a revetment he had constructed on the River Calder with stone pitching at 45-50° to the horizontal resting on a lower slope of loose stones at 25-30°. Several miles of these facings had lasted 7 and 9 years. He disagreed with Colonel Jones in that "*a comparatively small disturbance of the foundation … would destroy the equilibrium and the super structure would be overthrown”*.[31] He was inclined to favour a wall curved in section from 10-15° at the wall toe to 70-75° at the top, "*should the footing below low-water mark*".[32]

[30] It is unclear whether he is quoting V:H or H:V, although most probably he means 1V:4 or 5H as he is mostly promoting vertical walls.
[31] This perceptive observation is mirrored by discussion in section 3.5, see also Figure 3.8.
[32] Taken overall, it appears clear that Colonel Jones and most of the participants under-estimated the influence of wave reflections in lowering the 'no-disturbance' level for the rubble foundation. They also omitted to assess the influence of the mound on shoaling wave action towards impulsive breaking. This mistake continues to be propagated by many others including Vernon-Harcourt (1885), but partially corrected by Shield (1895). There is also significant confusion over the role of permeability in allowing inward pressures / flows vs trapping outward pressures. That said, some good points are made, especially Palmer on obliquity, and various others on setting the wall toe well below LW.

## 11.2 Russell (1847), with discussion

[John Scott Russell had been a teacher and experimenter at Edinburgh until 1844 when he moved to London and worked as a ship-designer at Milwall. In 1834 he had identified the ‘wave of translation’ – the solitary wave by observation on the Union Canal near Edinburgh. In 1836 he was appointed by the British Association to investigate the subject of waves. His report recorded observations on waves of the sea, tidal-waves, and on the effect of estuaries in modifying tides, waves generated in canals and other confined channels.]

This ICE paper and discussion (chaired by Sir John Rennie, ICE President) is primarily on wave mechanics, and on the form of breakwaters or seawalls.

Russell notes that "*... there is no general rule applicable to all cases*". He also speculates: "*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...*" Russell’s main purpose is to identify the key principles as: "*it is better to have rules with exceptions, than exceptions without rules.*" He then enumerates 12 'facts' or observations, although some are repeats.

Importantly, he distinguishes storm from swell waves, emphasising the role of swell waves in moving beaches and damaging structures. He speculates on a maximum wave height of 32ft (< 10m) in British seas. He presents a table of wave lengths (in feet); periods (in seconds); and velocities (in ft/s), although he does not describe the derivation of the table. In discussing marine structures, he tries to separate those like lighthouses that are "*incidentally exposed to the action of the wave*" from those like breakwaters that are intended to "*produce a direct effect on the waves*". For the first, he recommends a curved (trochoidal) form so that waves "*act equally over all points of the surface*" and "*will pass clear over without breaking*". When however trying to break the waves, he recommends a parabola.

When reflecting waves, he recommends vertical to 45° walls. He suggests that the wall height (above SWL) should be $R_c = H/2$ (probably where $H=H_{max}$). He notes that any (even slight) movement of the underlying foundation will remove restraint between blocks, allowing blocks to be moved by wave forces.[33] He also discusses the form and performance of parapet walls.

On wave loads on walls, he notes measurements of wave forces by Thomas Stevenson, from which it was concluded that a wave of 30ft (9m) might exert pressures of nearly 3720 lb/ft$^2$, equivalent to (only) 7.4kPa. In a reconstruction of this situation, Goda's method might calculate 50-70kPa, and Ito's ~64kPa.

In the discussion following the formal paper reading, **Russell** described a set-back parapet, and its effectiveness in reducing wave overtopping. He did however note that such a configuration demanded that the top of the slope be able to resist the wave loads, without the assistance of the weight of the parapet wall.

**Mr Murray** highlighted the advantages of managing beaches by groynes, citing successes at Sunderland where a few groynes had accumulated sand at the toe of the cliffs, halting previous erosion. On sea defences, he preferred to use mass armouring, allowing the power of the waves to re-shape random rubble rather than close pitching where "*the receding wave tore out the stones which had been previously loosened by the impact of the waves, or had been blown up by the air between the joints*". He ended his contribution by recommended shingle beaches as they "*always assumed such a slope as was best adapted for ...the locality and the force it had to resist.*"

[33] Again, see discussion in section 3.5.

**Mr Edwards** noted that many places did not show the same advantage from adding groynes and cited down-drift locations near Lowestoft and Yarmouth.

**Mr Simpson (ICE Vice President)** had recently constructed a pier (breakwater?) at Hartlepool to form a harbour of 11 acres (0.5km$^2$). The core of the breakwater was formed by a "*hard red marl … punned with 1/4 to 1/6 ballast*". This was then covered by stone and then pitching, so "*was quite impervious to water*".[34] On the issue of 'harbours of refuge' on the east coast, Simpson recommended many small ports rather than the "… *large harbours of refuge contemplated by the Government …*".

**Sir John Rennie (ICE President)** emphasised the need to consider the materials to be used, as well as the structure form and wave action. He contrasted use of sandstones versus granite or "*materials of greater weight*". He did not fully support Scott Russell's parabolic slope having had damage to such a toe 7ft (2.1m) below MLWS. Rennie's main contribution was however a description of the South Pier at Whitehaven[35] (with a cross-section) in a tidal range of some 25ft (7.6m), completed (probably) in 1834. This breakwater was 75ft (23m) across at the top, and 100ft (30.5m) at the base. The outer wall was 18ft (5.5m) thick at the base and 14ft (4.3m) at the top. The outer stone courses were two feet thick (0.6m), dowelled horizontally and vertically, and set / grouted by hydraulic mortar. The outer wall was further strengthened by counterforts 8ft deep (2.4m) and 6ft wide 1.8(m) at 15ft centres (4.6m). The parapet wall (approx. 2.5m wide and 3.7m high) was formed "*…of finely dressed stones, dowelled and bonded vertically and horizontally…*". A projecting cope gave a recurve effect to reduce wave overtopping. The roadway was paved by large ashlar blocks to a depth of 3ft (0.9m), bedded in hydraulic mortar.

## 11.3 Harbours of Refuge Commission (1847) – letters of dissent[36]

22 January 1846 - Symonds to Martin (Commission chair)

Expresses his "*unqualified dissent from the substance of the Report*". Earlier letters give reasons, but summarised here:

- Large area is not needed, harbour only used by war steamers, Post-office packets and occasional merchant vessels
- Harbour at Dover needs "*spacious eastern and western entrances to enable tide to scour it …"*
- Must be "*easy of access at all times of tide to vessels requiring shelter*".

Evident that harbour with only two entrances to the south will be difficult to access / leave. Concur with opinion of engineers that a close harbour will in times be destroyed by silt.

Evidence on mode of construction is conflicting. Theorists favour a vertical wall, but are not supported "*by a single practical experiment in deep water*". Symonds notes particularly difficulties in entering Valetta under NE winds making it difficult "*owing to the extraordinary agitation… created by*" the vertical walls of rock to either side of the entrance. He contends that "*no shipowner … would be willing to submit merchant vessels to the danger of entering so difficult a harbour as the one proposed*." Symonds favours the use of stone in a rubble mound as in Algiers anticipating that the vertical piers at Dover would cost £6-7,000,000[37].

16 July 1844 - Similar in letter from Symonds to the Secretary to the Commission for Harbours of Refuge.

24 January 1846 – Douglas to CE Trevelyan (Secretary to the Commission for Harbours of Refuge?)

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[34] This use of a low permeability core is very similar to use of chalk blocks to form the core of Stone Pier at Margate.
[35] See the paper by Williams (1878) discussed in section 13.6, also Figures 13.5 and 13.6.
[36] This is a series of letters summarising dissent by Captain Sir W Symonds and Lieutenant-General Sir H Douglas to the Commission's report, and why the Chair did not annex them to the report. There is much argument on procedures, and key arguments are repeated.
[37] In the event, the Outer Harbour at Dover was completed by S. Pearson & Sons from 1897-1909 for £3,500,000 – see Wilson (1919), sections 2.13, 6.4, and 13.12, .

Douglas objects to the form of construction proposed, and is clearly in high dudgeon that the Chairman will not include his dissent with the Commission's report. His letter is mainly on procedures, drawing analogies between the (the Chairman of the) Commission's behaviour and methods used in courts martial.

16 January 1846 – Minute of the Commission noting that "the members present have come to a decision that the individual opinion of a member cannot be allowed to be appended to the Report.

3 October 1845 – letter from CE Trevelyan to Admiral Martin on the procedure "*I am commanded by the Lords Commissioners of Her Majesty's Treasury to acquaint you that it was not their Lordships' intention that the usual course of proceedings should be departed from in this instance …"*.[38]

Further letters continue through 21 to 25 January reinforcing the main party's position, including a – Minute of the Commission noting that "*The Commission unanimously* (of those present!) *decide that … no dissent shall be inscribed on their Report"*.

To all the above is appended Sir Howard Douglas' dissent in 43 Articles and Annexes A to O.

**Article 1** – Asserts that the proposed modes of construction are "*untried upon any sufficient scale to warrant their present adoption …*"

**Article 2** – Asserts that building an upright wall in 7-8 fathoms (13-15m depth) is "*novel in theory and never … proved in practice…*". He draws comparisons with sloping breakwaters at Plymouth and Delaware.

**Article 3** – Continues the support for a sloping breakwater, adjucing support from many others (although quite selective). He quotes Alan Stevenson "*an upright wall … an experimental measure … work of the utmost difficulty …a force would be developed by the collision of the wave with the wall … will surpass any which has been experienced on the face of a sloping breakwater."* Douglas had always objected to concrete (in a wall). He cites reported success in use of concrete blocks at Cherbourg, but then "*an important failure … employment of concrete as a substitute for stone … has been abandoned by the French engineers."* He claims further that concrete at Algiers has deteriorated allowing the mass of the breakwater to settle.

**Article 4** – Claims that Professor Airy was "*not aware of any case in which a perfectly upright wall has been built in the open sea … (to depths of 13-15m)… admit of a doubt whether it would end in failure…*"

**Article 5** – Cites General Sir Harry Jones (1842) paper to ICE in April 1842. Douglas concludes that a majority of speakers (7 to 2) in the discussion to that paper, favoured armoured slopes over the vertical wall.

**Article 6** – Here he disputes the citing of the pier at Kilrush as a suitable example of an upright wall, particularly its use in supporting such a wall at Dover.

**Article 7** – Notes the engineers consulted by the Commission, including: James Walker, George Rennie, Capt Denison, Sir John Rennie, Lt-Col Harry Jones, W Cubitt, Charles Vignoles, J.M Rendel.

**Article 8 -** Quotes James Walker as suggesting pre-cast caissons of some 90-120m length, 20m width to be towed by steam tugs and sunk in position at Dover. Douglas then indicates that Walker favoured a long slope for exposures like that at Plymouth. He then however diverts to a discussion on repairs to the foundations of Westminster Bridge! [39]

**Article 9** – Cites George Rennie as "*deprecates the upright wall as impracticable and dangerous …*".

**Article 10** – "*Capt Dennison is for a vertical wall formed of hexagonal prisms of concrete…".*

**Article 11** – Sir John Rennie deprecates "*all systems of caisson … and proposes the adoption of the principle observed in the breakwater at Plymouth*."

[38] So no dissent from the majority view was to be published.
[39] In the various discussions on the use of "caissons", it is interesting (if somewhat disheartening) that no mention is made of those used with some success at Tangier, see Routh (1912), rather justifying that author's ephithet as a "lost outpost"

**Article 12** – Colonel Jones favours a sloping breakwater with an upright wall on it.

**Article 13** – Mr Cubitt also favoured the form of breakwater at Plymouth.

**Article 14 -** Mr Vignoles suggested a sloping breakwater of concrete cubical blocks to low-water, surmounted by a vertical wall.

**Article 15** – Notes that Mr Rendel is claimed as an advocate of a vertical wall formed by brick blocks. In his examinations by the Commission on 19 June 1844 and November 1845, Douglas suggests that Rendel might renounce that view depending on availability of materials.[40]

**Article 16** – In summary, Douglas argues that only one of eight engineers who submitted plans favoured the vertical wall.

**Article 17** – Douglas then claims that those supporting the upright wall included: Professor Airy, Professor Barlow, Major-General Sir J Burgoyne, Sir Henry De la Beche, Mr Hartley, Major-General CW Pasley, Captain Vetch, M Reibull, Mr Brunel, Mr Bremner.

**Articles 18 to 27** – Douglas then attempts to gainsay each of the submissions from those in 17 above.

For Professor Airy, whilst accepting his theory that an upright wall is always preferable to a slope, Douglas notes Airy's response that "*building an upright wall in the open sea (in 13m) is so far an experimental measure, that no such work has ever been executed.*",

He notes Professor Barlow's[41] "*mathematical investigation*" but quotes that he "*avows that he has not sufficient practical knowledge or experience to enable him to speak confidently on the subject…*".

Major-General Sir John Burgoyne in supporting the vertical wall had [correctly] deduced that deep water waves had "*very little, if any, forward motion, except where it breaks*".

Sir Henry De la Beche's words on wave impacts at cliffs (with approaching beach slopes) are used by Douglas to again cast doubt on the geologists' support for a vertical breakwater at Dover.

Similarly, Douglas casts doubts on Mr Hartley's support for the vertical wall proposal, quoting back excerpts from his answers to particular questions[42].

Noting Major-General Pasley's support for the vertical wall, Douglas again quotes excerpts from discussions on the comparative merits of sloping or vertical walls.

Douglas again quotes Captain Vetch as agreeing that he does not know of an example of a simple vertical wall in seven fathoms, although Vetch does suggest they could be formed by caissons.

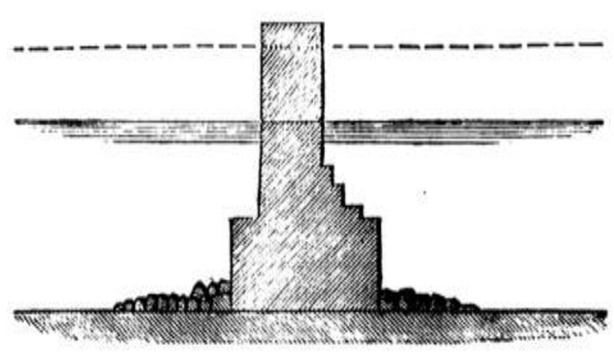

**Figure 11.1 Reibell's vertical wall**

M. Reibell had sketched a vertical wall as in Figure 11.1, possibly to be preferred to the sloping form used previously at Cherbourg. Douglas then quotes doubts by Captain Washington as to whether such a wall could be constructed "*in the open sea*", and whether it would "*survive the first storm*". Douglas also complains that there would be no room along the crest for batteries or for traffic.

[40] This is somewhat confusing given the absence of rock close to hand to form a rubble mound at Dover!
[41] Professor Peter Barlow, Royal Military Academy, Woolwich.
[42] Presumably from one of the Commission's sessions)

Douglas disparages an opinion in favour of the vertical wall by Mr Brunel as "*summary, off-hand*", speculating that Brunel's comments were "*given with reference to sea-walls*".[43 44]

Mr Bremner's support for vertical wall solution, particularly constructed by caissons, termed "*utensils*" is again disparaged with the reference to difficulties then being experienced with caisson foundations for Westminster Bridge piers.

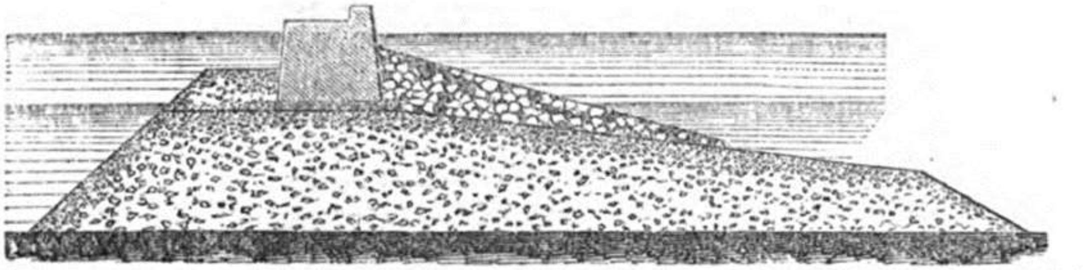

**Figure 11.2 Douglas' long-slope at Cherbourg**

In Articles 29-38, Douglas discusses at significant length the breakwater at Cherbourg, particularly in its evolved "*long-slope*" form, see Figure 11.2 and drawing comparisons with Plymouth breakwater. He likens this to "geological" beaches and cliffs, claiming that these only lead to the generation of smaller (granular) material at long gentle slopes. He draws some conclusions from the experiences at Cherbourg, some of which appear entirely obvious, and others that might be perceptive if they were not muddled by other assertions or conclusions. For rock slopes Douglas concludes that the rock size and slope need to be in equilibrium with the (storm) wave action, and that a compound slope, steeper over lower and upper parts, may suffice if the part around the water-line is sufficiently long. He concludes however that the middle slope had to be "*far longer than that which was originally designed*", implying a requirement for much greater volumes of rock than budgeted.[45] He notes that small blocks have little stability, and that "*very large blocks should be placed towards the top of the work*", although his argument of the effect of submerged buoyancy is not entirely logical, ignoring that the effect applies to all submerged blocks, if even only by run-up.

On replacing rock armour, Douglas records that use of concrete blocks at Cherbourg "*entirely failed…the blocks of concrete having broken to pieces*", although he gives no detail of the type of cement used, or the mix design! There is therefore relatively little of utility that can be learnt from this observation.

Douglas then rehearses disagreements between Mr Emy and Mr Airy on the form of wall: vertical versus concave, although he appears to find that (in part) they agree. He also re-states (again) his objection to replacing rock by concrete, or masses of brick.

On the harbour layout, Douglas argues for a second (eastern) entrance, which successfully appeared in the Coode 1897 design constructed by Pearson.

## 11.4 King-Noel, William (1848) on Harbours of Refuge[46]

The author starts his paper with statistics from a House of Commons Committee reporting in 1836 on vessels, lives and cargo lost. During 1816-18, 1114 vessels were lost, and during 1833-35, 1573 vessels. This was

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43 Perhaps at Dawlish.

44 Douglas' criticisms here are fairly speculative with little or no evidence being advanced in support of his arguments, but much assertion. Douglas is correct that, in the absence of concrete technology to form large blocks, and of diving techniques to assist placing foundation blocks at depth, then construction of a vertical wall breakwater in seven to eight fathoms of water (13-15m) would certainly be difficult. But the success of Pearson's construction of the Coode design for the Dover outer harbour (Wilson, 1919) demonstrates that not only would it have been eminently constructable, but very stable when completed (see the Dover case study in Chapter 6). Indeed, many of the inherent flaws of the wall-on-mound solution were later demonstrated by failures at Alderney, Wick (Chapter 6) and elsewhere.

45 This may well be a reference to the then continuing construction of Plymouth Breakwater.

46 William King-Noel, 1st Earl of Lovelace, was not a civil engineer, but was awarded a Telford Medal by ICE for this paper, and was then elected as an associate of ICE It is not clear to what extent this paper is personal (some clearly is), or whether it constitutes some form of informal feedback to civil engineers from the various House of Commons Committees on Harbours of Refuge, primarily on the idea of a new harbour at Dover.

equivalent to annual losses of 894 lives, and £2.8 million of cargo. The Committee assigned ten causes, at ninth was "*Want of Harbours*". He then noted that another committee took an interest in 1843, during which time "*circumstances had occurred to impair the entente cordiale … between England and … France … possibility of rupture followed by a destructive war of naval aggression … the continuance of peace … not to be so secure as formerly*".

The Duke of Wellington, Lord Warden of the Cinque Ports, had frequently considered the effect of a war with France, and had "*pronounced himself strongly in favour of harbours of considerable extent, capable of sheltering some hundred sail of shipping …"*. The Duke drew attention to changes between "*a naval war on the old principle, and maritime war such as would be resorted to … since the adoption of steam.*"

The author then recommends "*two separate objects … to be combined as much as possible: safety from the weather; and defence against the enemy*." For the former, he reports that "… *it has been proposed to accomplish by means of floating breakwaters, for which … there are numerous ingenious expedients …*". He then cites a number of the ideas that have been put up to the committee with, in some instances but not all, reasons against their adoption. He starts with the concept of large frames or gratings from 18-30m long to be anchored or secured by Mitchell's screw moorings (iron piles formed with an augur screw to the lower part). He does however acknowledge the "*great difficulties*" in how the gratings might be laid out, to provide adequate shelter, but not to collide and cause damage to themselves or the moored vessels. He cites proposals for such gratings by Captain Taylor (tested at Brighton); by Captain Pringle resembling huge farm gates; by Captain Sleigh using floating caissons; and by Major Parlby using a gigantic band of artificial reeds in the form of hollow floating trumpets made of coir. [47] But then in a simplified analysis of anchoring loads he condemns all the floating devices, and notes that it was "*not therefore surprising that these projects met with little encouragement*".

In April 1844, Sir Robert Peel's Government required a Commission to report "*whether a harbour of refuge in the Channel was desirable, … and what should be its site*". Any such site should be assessed against three attributes:

1. *Accessibility at all times of tide to vessels requiring shelter from stress of weather*;
2. *Station for armed vessels … both for offence and defence*;
3. "*Possess facilities for securing its defence in the event of attack by the enemy"*.

The Commissioners worked rapidly, and reported on 7 August 1844. "*They arrived at three distinct conclusions, recommending … a harbour of 520 acres at Dover; then one at Portland, next Seaford; and lastly Harwich.*"[48] The Commission, as reported by the Earl, then fell to obsessing about siltation, and accumulation by longshore drift, which they do generally (but not always) treat separately. Two wild-cards were introduced with a minority recommendation for a harbour at Dungeness, and another for an offshore harbour sheltered by the Goodwin and Brake sands, proposed by Captain Vetch who anticipated fixing the sand by an embankment formed by driving iron rods into the sand 0.3m apart "like a large bird-cage"! He "*imagines the Goodwin Sands would in time become an island of 8000 acres …* ". The author claims to the 1843 Committee that "*the idea of enclosing the small Downs … had been also entertained by Sir John Rennie*".

Next, turning to Portland, the Commissioners recommended a breakwater 2km long sheltering 1200 acres. This recommendation appears to have had many advantages and few opponents. The Earl then permits himself a sideways diversion to discuss Harwich, and the mobility of its entrance. This led him to rehearse arguments made to the Commission on siltation and/or accumulation of shingle, partially supported (or confused) by measurements. M. De Cesart had calculated rates of 30,000 m$^3$ of shingle "*are annually washed into the ports of Dieppe and Treport on the opposite side of the Channel*", and Captain Peat of the Folkestone Coastguard had seen "*from 100 to 500t of shingle heaped up … in a single gale*". Sir Henry De la Beche had reassured the

[47] Honestly, the fuller version is no less daft!
[48] Yes, the noble Earl appears not to be able to count!

Commission that, provided that the projected harbour entrance was some 200-300m from the shore, "*it would be a long time*" before the shingle drift would block the entrance. It was probable that a breakwater projecting some 550-640m into 13m depth would form a 'ness' "*behind which the shingle might accumulate for ages*".[49]

Turning to siltation, being the carrying into a quieter area of fine material in suspension, Captain Vetch (again) quoted Smeaton as suggesting that of order 3000m$^3$/year had accumulated in Ramsgate Harbour. More usefully, Captain Washington made measurements at Dover where he reported in March 1845 on measured suspended loads from which he computed a limiting siltation of ~150mm per annum. He revisited the measurements later that year and reported in November 1845 on further measurements indicating about half of that, suggesting that works on the cliffs for the railway may have increased turbidity for the first measurements. The Commission had concluded that such siltation could easily be tolerated, even if dredged at a cost of 8d/yd$^3$ would only amount to £7000 per annum.

The Earl then turned to the form of breakwater, branding a vertical wall of such dimensions as "*in a great measure experimental*", even though that appeared to be the advice for Dover. Against that, for "*breakwaters composed of loose stones ... which have more or less answered the purpose for which they were designed, yet it has always been after long delays, great cost, and heavy and constant repairs*". He notes that the breakwater at Cherbourg was still unfinished after sixty years, and rehearses some of the tribulations of Plymouth Breakwater, although it would appear from the language that the noble Earl does not support the views of Mr Rennie and Sir John Rennie in persisting with the "*long slope*". It is probable that his view is crystallised by: "*it is difficult to comprehend how the substitution of solid masses of masonry, skilfully put together and lowered into their appointed beds by powerful and exact machinery, and the arrangement of them by divers at 30ft and 40ft below the surface, instead of dropping in huge cyclopean fragments to shake themselves into places as they best may, can be called a retrograde step in science*."[50]

At this point, the author runs through many of the problem sites in Ireland described by Colonel Harry Jones (1842), although apparently now with new data on damage. He again rehearses the tribulations of Plymouth Breakwater with details of quantities of additional stone needed after storms in 1819, 1824, 1830, and 1838. Clearly the builders had returned to the Admiralty requesting '***more***' on rather too many times.

One deduces that the noble Earl had some significant regard for Lieut.-General Sir Howard Douglas who disagreed with his Commission colleagues' conclusions, but whose opposition led to "*a very interesting discussion, eliciting many curious views.*" In particular, the Earl reports discussions between Sir Howard Douglas and Sir William Denison (who had been directed to confer with Colonel Harry Jones). Both parties were skilled at verbal fencing on hydrodynamics of waves at sloping or vertical walls, reported in some detail by the author to more than a page of the proceedings, and appearing to result in a points win for Sir William Denison. [51]

Captain Vetch (again) supports the use of vertical walls, and distinguishes between oscillating wave motions (better known now as 'pulsating') distinguished from breaking or percussive (now termed impulsive) with some perceptive observations from a landing at Scarnish, Tiree.

Professor Airy was also subjected to questioning on when (and how) waves break. He describes relatively clearly how impulsive pressures may be transmitted within jointed masonry to force the face-stones out, but confuses the issue by speculating that oblique attack might be more dangerous than normal attack! Professor Airy however clearly impressed the Earl, persuading him against "*the argument* (for) *long slopes*".

[49] I assume that they meant ' in front of'.
[50] From the lag of some 170 years, the point could hardly have been more correct or to the point. The caveats are obvious, and perceptive.
[51] In the course of the discussion, Sir Howard Douglas postulates a simile remarkably like Smoothed Particle Hydrodynamics, some 150 years ahead of his time?]

The author then turned to how the vertical wall might be formed. Sir William Denison proposed to manufacture concrete blocks (the solution indeed adopted by Coode / Pearson in 1897, see Wilson, 1919). Mr Walker proposed building portions of wall in caissons at Portland and floating them to Dover[52]. Mr Rendel proposed to unite bricks by cement or vitrification into 8t blocks, and to use divers, an approach that it appears the Commission favoured.

Sir William Symonds objections to Dover on the basis of insufficient holding power for anchors was rapidly countered by experiments by the *Blazer* where 8 runs tested the holding quality of the bay, and found it "*far better than ... he had anticipated*". In a discussion on the width of entrances, the author notes "*that there may be steam-tugs ... to facilitate access*".

The noble Earl then returns to the agonising discussions over the form of structures with (considerably) more on form and composition. He notes that the Americans had decided to use rubble mound breakwaters to protect Delaware Bay, but then noted "*after perusing the documents issued to their countrymen ... it must be owned that the reasoning appears inconclusive and its framers take no notice of the disasters that have befallen the great French work since the Peace. Besides there the principle has actually been abandoned ... and a vertical wall has been erected on the rubble.* " [53] Professor Airy in commenting on this mode of construction "*pronounced it to be the worst.*" The Author continued waspishly: "*A slight examination shows that the eminent French Engineer who resorted to it was not his own master; he made a merit of necessity ... to obviate the consequences of having adopted an erroneous principle in the beginning ...after 40 or 50 years' perseverance had been found inadequate.*"

Continuing the debate on structural form, the author then revisits the relatively recent paper on breakwaters by Colonel Harry Jones (1842). More examples of failures of rubble mounds are adduced concluding that for waves "*it is only after a most destructive and grinding course that they finally close their stormy career under the ceaseless billow.*" One can conclude that the noble Earl was not enamoured of the rubble mounds of the day! However, conclude he does not, and embarks on a long debate of the effects of "*flot du fond*" as discussed by Colonel Emy in his 1831 book "Du Mouvement des Ondes, et des Travaux Hydrauliques Maritimes "[54] In any event, many of the wilder notions are debunked by the author, citing especially Airy and Scott Russell. He also cites measurements by Stevenson at Skerryvore, later published by Stevenson (1849).

## 11.5 Stevenson T (1849) – communicated by D Stevenson[55]

In considering the problem of wave forces, Stevenson writes: "*In forming designs of marine works, 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. I shall explain the construction of this simple self-registering instrument…*" a self-registering device (termed a **Marine Dynamometer**) to measure wave forces on a circular plate. He describes the device in Figure 11.3 in which the force applied by the wave to this plate is resisted by internal springs, the degree of compression being recorded by the travel of leather rings that slid along rods connected to the circular disk, indicating how far those rods have been pushed in by the wave force against the (calibrated) spring. The maximum wave force during a storm could therefore be deduced by the position along the graduated rods of the leather ring, after which the ring was pushed back along the rods to re-set it to zero. In siting the device, and in considering the results, the author noted that it was "*... almost impossible to receive the force unimpaired, as the waves are more or less broken by hidden*

[52] Had he recalled those at Tangier, see section 2.3?
[53] This must be a reference to Cherbourg where the original 'cones' had been abandoned, replaced by a long rubble slope with wall atop.
[54] In the discussion "flot du fond" takes on a portmanteau character, appearing to mean whatever the author wishes it to. It probably means 'near-bed wave orbits getting constrained and moving sediment', although it may have a connotation of wave shoaling and approaching impulsive motions.]
[55] This nine page note including pictures and tables describes the development and use of a wave force measuring device, and presents tables of wave forces measured. Most of this material is repeated in Stevenson (1874).

*rocks or shoal ground before they reach the instrument.*" He also noted an attempt to observe wave heights on a graduated pole, but "*... the poles soon worked loose ... and disappeared.*"

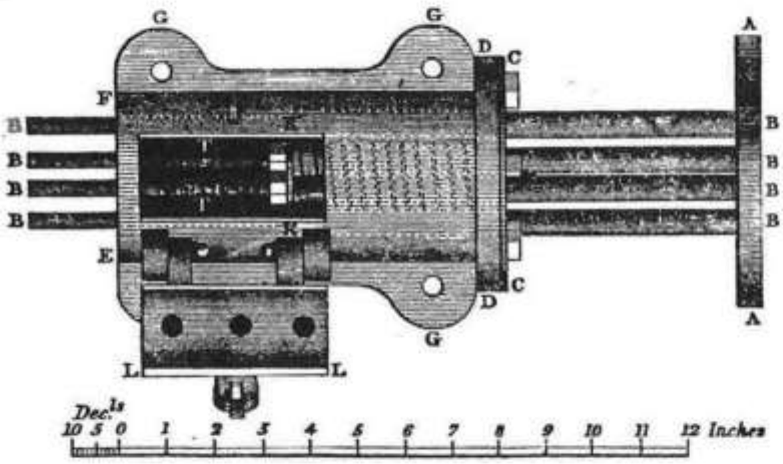


**Figure 11.3 Stevenson's dynamometer (wave load plate to the right)**
*From Stevenson T (1849)*

The first observations in 1842 (reported in a table at the end of the paper) were made on Little Ross Island off Kirkudbright (Solway Firth). Then he deployed the device on Skerryvore off Tryee from "*... April 1843 till now*"[56] . Further (?) measurements were made on Bell Rock from 1844. The largest force measured (at Skerryvore) was 6083 lbs/ft$^2$ (292kPa), but on Bell Rock only 3013 lbs/ft$^2$ (144kPa). He notes that "*there can be no doubt however that results higher than this will be obtained*." He then discusses apparent reductions in impulsive loadings, having dropped a cannon-ball onto the dynamometer sensing plate, although he gives no data from those experiments. Again, of rather less utility, he cites instances of blocks of stone moved by wave action, and calculates wave pressures to raise spray to a height of up to 106' (32m). He notes the amused satisfaction of local residents at Barra who had advised that the sea "*... would reach the storehouse...*" used for construction of the lighthouse, much denied before the event by the workmen, but with movement of stones of upwards of 40t. In the accompanying tables, the author lists individual maxima by location and date (from April 1842 to March 1845).

## 11.6 Scott M (1858) on Blyth, and wave theories [57]

On Blyth, Scott starts "*... a few years ago, a company of enterprising gentlemen ...obtained powers for improving the harbour.*" The author was involved in constructing the breakwater, following a design by James Abernethy (see later intervention in the discussion). The design envisaged use of timber piles and planking retaining rubble infill. Much of the later part of the paper is devoted to 'advertising' this system, emphasising the advantages over constructions at Alderney, Holyhead and Portland (again, see the later discussion).

Before turning to the design / construction of Blyth breakwater, Scott rehearses a number of 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 or Scott Russell. He enumerates ten points on waves of oscillation, amongst which he draws out a particularly important conclusion: "*... the destructive energy is less against a (vertical) wall ... than against a wall built upon a slope. When the foreshore causes a wave to break upon the wall, the*

[56] 'Now' is here assumed to be later than the last entry in the Table, so after April 1845, and perhaps up to sometime in 1849.

[57] At first, this paper would appear to be 'Site-Specific', but the paper itself, is more generic and rather speculative. The discussions are certainly wide-ranging, indeed aimed at anywhere but Blyth, hence its categorisation under 'General'. Scott has some sensible but occasionally muddled comments on wave action, and some useful remarks on blockwork walls. He then however seeks to lead the reader and his audience at ICE astray by advocating timber wave screen with rather little supporting evidence.

*destructive effect is greatest.*" Following those remarks, he 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. He also recommends a vertical face on the inside so that vessels can berth. 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 far, so good, but sadly this clarity is then substantially undermined by a major under-estimate of the water depth needed for the stability of a rubble foundation in front of a vertical wall. Scott sketches a vertical wall on a mound, and "*… believes at Cherbourg, Plymouth and more recently at Alderney, Holyhead and Portland, that a mass of pierre perdue will lie permanently at a slope of 1 to 1.5 … up to 12ft* (3.7m) *below low water.*" A further significant lack of clarity is introduced by the remark about Alderney: "*… Mr Walker first deposited from barges a mass of pierre perdue which reached to within 12ft of low water; but then the angle of the vertical wall up to low-water appears to have been filled in …*" without in any way identifying whether this was by design, by accident, or natural causes![58] Scott later notes that where the wave break on a rubble slope this may lead to abrasion of any staging, and of the stones themselves, citing broken piles at Holyhead and "*… worn and rounded …*" stones at Portland.

On placement of blocks in a wall, Scott perceptively notes that: "*… their weight* (meaning their resistance to movement) *is greatest when laid in a horizontal plane … pressure of super-incumbent stones is greatest in a vertical wall … it follows that settlement is fatal to the stability of a vertical wall … for if one course sinks, all the power of resistance due to the weight of the super-incumbent masonry is lost.*" He notes the role of air compression within blockwork joints in causing damage where it "*… communicates the pressure through all neighbouring interstices … almost instantaneously, and thus blows it up.*" On water pressures, he is less clear, omitting any reference to the importance of the internal permeability. His recommendation that "*…the faces of works exposed to the sea should either have the joints sealed to prevent the entrance of air, or the work should be so open as to allow the air to pass out with facility…*" is therefore less than clear, as it omits the importance of the porosity / permeability of the internal space / fill.

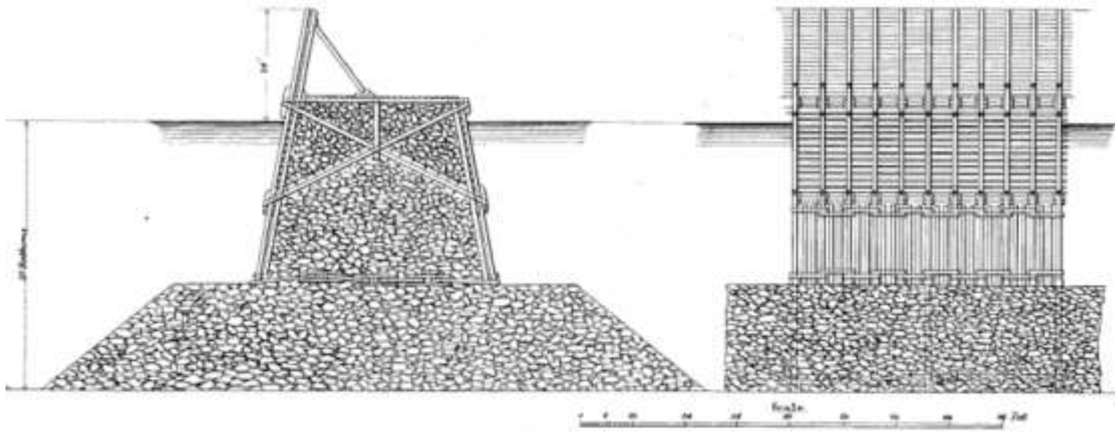

**Figure 11.4 Timber crib breakwater**
*After Scott (1858)*

Returning to his own (?) designs / construction, he discusses breakwaters formed by filling a timber frame with stone, noting that the use of a frame (tied across the base by a chain?) avoids the need to drive piles. He

[58] He also omits to mention that the foundation level for the wall at Alderney was successively moved downward as construction advanced, see section 2.7, 11.2, and 13.4.

contrasts the slow speed of works at Alderney, Portland and Holyhead with a construction rate at Blyth of 130ft (40m) in five days, although he does acknowledge the "*comparatively small scale*". He then claims that a 10 fathom (18.3m) section could be constructed at £70/ft run, against Alderney at £190/ft, and Portland at £150/ft, but see later comments in the discussion. For Blyth, he also claims "… *that the stability is double the greatest force which could be brought to bear upon it*"[59] In comments to a later discussion on the speed of construction, he notes that other breakwaters taken down to the seabed have required helmet divers or diving bells, both subject to disruption by tidal currents and/or swell waves. He notes that use of prefabricated frames, Figure 11.4, then filled by stone, removes the requirement for use of either divers or bells.

Under the headings of "*Durability and Cost*", Scott accepts the potential for attack of timber elements by *Terodo* or *Limnora*., although he asserts that creosoting, "*when properly done ... will preserve timber for many years ...*". He then however implicitly acknowledges that a timber structure might be regarded as temporary, so could be faced by masonry, with the timber frame providing "*an excellent staging from which to build the stone facing.*"

On the topic of moving large (concrete?) blocks, Scott describes use of ingenious "*hydraulic machinery*" to allow movement of powered crabs and hoists to assist move large blocks under diver control, all powered from his own (patented) hydraulic accumulator![60] The presentation of the paper was illustrated by various diagrams, and a number of models.

## 11.7 Discussion to Paper 991 by Scott to ICE, pp91-161 (1858)

In the extended discussion to this paper, running to some 70 pages, there would appear to be significant influence of the *Report from the Select Committee on Harbours of Refuge* published in June 1858, or perhaps more to evidence that had been, or might have been, given to the Select Committee. The (sometimes ill-tempered) discussion also appears to have been used by a number of members to plug their own design ideas, to advance claims, or counter-claims, perhaps even to settle scores! The record of the discussion is surprisingly lively, and includes some encouragingly perceptive contributions.

**Mr TR Winder** touched on four approaches to building breakwaters described to the Select Committee, although he primarily discussed progress (on the Admiralty Pier at Dover) where diving bells were used to prepare the foundations for a full height wall. Captain Vetch regarded progress at Dover as "*slow and expensive*", although James Walker (later ICE President) defended the "*extreme difficulty of the work*". Opinion was therefore generally against this approach. Winder however believed that lessons learnt early in the job held out the prospect of quicker progress as the job advanced. He postulated construction using 50t (concrete) blocks, with potential costs described at length in the discussion. A figure showed the possible construction approach using overhead travellers on staging (remarkably similar to those used later in the outer breakwaters at Dover, see Wilson (1919). Winder cited in support the construction of a small pier at Rye composed entirely of a single column of 10t concrete blocks (2.7m across, 1.2m x 1.4m section).

**Mr Murray** discussed (at some length) the apparent effect of training walls in reducing tidal range in estuary rivers, claiming that tidal high water on the River Wear was moved two miles seaward when the second pier narrowed the river entrance. He also claimed that Mr Shout had used timber frames to extend a pier by 600m on the Wear more than 70 years previously.

**General Sir Harry Jones** mused on the mechanisms for damage at Cherbourg and Plymouth, claiming that "… *it was well-known that the violence of the sea did not extend more than 15ft* (4.6m) *below low water…*" He felt that too much stone had been used at Alderney and at Plymouth than was necessary, speculating that

[59] But without showing any part of those calculations!
[60] No mention is however made of any potential dangers from motion of large machinery along or across staging above the works, controlled remotely by divers out of communication with the surface.

"*…if a cheaper and equally durable system were adopted, a greater number of these essential works could be undertaken.*" [61] He continued by suggesting that the harbour at Dover could be formed by cutting "*away the materials of the adjacent cliffs … to form a simple embankment in the sea … in same way as a railway … on land.*"[62]

**Mr Rennie** noted that the breakwater at Blyth was only 20ft (6.1m) high and constructed in very shallow water. He then remarked that the Western or Old Mole of Genoa had been formed by timber chests (caissons) filled by stone and cement, onto which a superstructure of 18ft (5.5m) height was added. He noted the failure of the timber cones at Cherbourg, and then noted the "*destruction … of the mole at Tangier built by Sir Hugh Cholmley in 1786 …*" citing "*An Account of Tangier by Sir Hugh Cholmley, Bart, 1787.* "[63]

**Mr Cooper** suggested that the paper referred partly to a hypothetical harbour in rather deeper water than at Blyth. He noted that entrance piers to the inner harbour at Dover had been formed by driving piles into the chalk and filled by stone, but that the Admiralty Pier was formed by large concrete blocks faced by granite.

As himself a member of the Harbours of Refuge Commission, **Mr Coode** was not able to express his views on many issues. He could however correct costs given for Portland, where even the most expensive section had cost less than £120/ft (£390/m).

**Mr Beardmore** noted the differences between structures for shallow seas versus those for deep water. "*Timber was a convenient material … if it was used in proper situations*" citing examples in Netherlands and France, but none were "*deep-water piers*". In deep water, he commented on timber that "*… the less it was used … the more successful would be the result.*" "*The reason was obvious because … a sea barrier which was absolutely rigid must have a weight superior to the momentum of the greatest mass of water striking against it.*" He also suggested that the construction rates suggested by the paper were far too optimistic, and not supported by recent experience at Dover.

**Mr Wells** described a breakwater design using creosoted timber planks supported on wrought iron piles on Mitchell's screws (shown in Figure 14.4).

**Mr Murray** noted that a Harbour of Refuge required considerably less shelter than a dock where vessels loaded or discharged cargoes. He again restated the general acceptance of the '12ft below LW' rule, then discussing many alternative sets of prices, many aiming to suggest that the form of construction being used at Dover was too expensive. He noted success in rubble-mound construction at Marseilles, where larger stones had been reserved for the outer surface, rather than lost in *pierre perdue* as at Portland and Holyhead. He suggested that 3$^{rd}$ class rubble (<0.5t) could be used from 36ft to 22ft below low water (11 to 6.7m), then 2$^{nd}$ class rubble (0.5t to 2t) up to 12ft below LW (6.7m), and 1$^{st}$ class rubble (2-5t) up to LW. This was armoured by 5-9t blocks laid "*with a long slope*". At Algiers, the armour was 22t concrete blocks, at Marseilles and Cette they were 25.5t, and at Cassis 20m$^3$ (46t?).[64] He then used cost estimates for such construction to suggest significant cost savings vis-à-vis the construction at Dover.

**Mr AJ Robertson** explained that the Author's costs for Portland were calculated from the engineer's reports to Parliament. He also noted the use of convict labour, then running through the relative contribution to costs.

**Mr Coode** referred to his evidence in 1857 to the Select Committee of the House of Commons, particularly on the (comparative costs for) use of convict labour, he "*much doubted whether … any pecuniary saving had resulted from the employment of convict labour*".

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[61] Oh, the touching belief in magic!
[62] The potential erodibility of (unprotected) chalk does not seem to have been factored in.
[63] This a particularly curious mis- (?) recollection given that Routh (1912) describes the demolition of the Mole at Tangier in 1683-1684, some 102 years earlier, see discussions on Tangier in Chapters 2 and 13.
[64] Interestingly, Murray appears to consider only hydraulic lime, with or without pozzuolano, in the 'concrete' blocks. Portland cement is not mentioned.

**Mr Mallet** noted the effect of a wave reflected from a vertical wall in scouring out the foundation, citing significant scour (1 to 1.25m deep) in front of a breakwater at Kingstown Harbour, and a smaller scour trench due to overtopping on the lee side. He therefore always recommended use of a suitable apron of concrete or stone to protect the toe of such walls. He continued with some (remarkably perceptive) comments on the effects of impulsive pressure on the removal (outward) of blocks in a wall, although the use of the term "*quaquaverse pressure*" is rather obscure! He notes that impulsive pressures are transmitted in through any joint or fissure at the speed of sound in water (~1350m/s), dislocating the joints and tended "*... to punch out gradually, or suddenly, prismatic blocks in a direction opposite to the stroke of the wave.*" He then cites removal of granite blockwork on the pavement to an Irish seawall under impulsive downfall pressures. He noted the effect to be greatest on open irregular rubble and least on deep-bedded ashlar.

**Mr Abernethy** said that the Author had "*selected the pier at Blyth ... as a text for very extended remarks on breakwaters generally. There was nothing in the details of the construction of the breakwater at Blyth ... deserving of the attention of the Institution*"[65]. Abernethy noted that the form of construction suggested came about primarily due to a (very local) shortage of (appropriate) stone.

**Sir John Rennie** had previously surveyed this site, at which the late Mr Rennie had proposed in 1814 to erect a stone breakwater at a similar cost as the cited works, so no significant saving. Sir John then considered (and rejected) whether the suggested timber frames might be suitable at the deeper and more exposed sites being considered. He discussed Kingstown, Howth and Plymouth Harbours. He noted that stone delivered to the latter was carried in 80t loads by sailing ship, yet in a single week in May 1816, 5330t were delivered[66], and in the whole of 1816, 332,400t.

**Mr Hawkshaw (ICE Vice President)** noted that the cost of the breakwater at Holyhead had been mis-stated at £200/ft, and had not exceeded £160/ft. He did not however believe that any Member of the Institution would be persuaded "*... that any one system of breakwater could be universally adopted ... The form of a breakwater must be governed by the local conditions.*" He did not believe that the suggested method of forming a breakwater could be applicable to Holyhead or Portland or other exposed sites, and felt that costs could not be usefully compared between different sites.

**General Sir Harry Jones** continued his earlier comments by noting that Harbours of Refuge did not, *per se*, require the same standards of protection, and it was wasteful to include large parapet walls, which of themselves caused wave action to fall on the breakwater crest. The deletion of these parapets would reduce costs and "*... the sea would go quietly into the harbour without disturbing the vessels riding there*." [67]

**Mr Parkes** returned to the '12ft below LW' rule, noting that he had seen a site where 18ft would have been required, in any case "*the depth varied in different places.*" He sought clarification from those working with waves as to what extent wave velocities would be changed (increased?) by positioning a reflecting wall on top of a mound, or whether the wall toe should be set 1-1.25m below the undisturbed equilibrium level. In any case, he doubted whether timber frames could easily be handled at the suggested sites of Portland, Holyhead or Alderney.

**Mr Bidder (ICE Vice President)** 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 as to the proceedings…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.*" He had referred to several Reports of the

65 A sentiment with which this author can agree.
66 The 5000t+ was probably the peak summer rate.
67 The General appears to have an optimistic blind-spot on wave transmission!

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.".* At Dover, he noted that a slope would not be applicable, but they were however appropriate at Holyhead and Portland. The costs of Dover, now at £415/ft, would require it to be abandoned. He hypothesised a construction period of 50 to 100 years (based on the then rate of progress), suggesting that "*… the great object … must be to devise some other and simpler system of construction for works of this kind.* " Bidder continued in an attack on James Walker (past President of ICE) who had signed the report of 1845 stating that the costs of a vertical wall or rubble mound "*would be nearly identical*" yet the pier at Dover was costing £415/ft; those 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*". Bidder 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.*" He noted a surveying error on depths at Portland of 7'6", increasing costs by ~10%, then turning to the costs (or not?) of the convict labour at Portland. He believed that the quoted cost of £930,000 should be increased to £1,500,000 when true costs of labour and interest charges were included. As well as the construction costs, or their accounts, he also criticised the entrance positions and widths, and the (expensive) form of the roundheads.

Bidder then turned to the Harbours of Refuge on Alderney and Jersey, the former being "*nearly valueless*" and that at St Catherine's offering "*scarcely shelter for a few fishing boats*"[68]. At Holyhead, he avers "*an unfortunate error in the original work; the plan having been built concave in plan instead of convex towards the sea … nearly 200 acres more of harbour space would have been obtained at the same cost.*" 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 …*"

On Portland, **Mr Coode** reminded the meeting that the project had been open "*to the competition of six of the most eminent contractors in the kingdom, and upon that the works were let."* The use of convict labour appeared now to offer only a small saving. The entrances had been decided by the Commission in 1845 to assist leaving the harbour under northerly winds. The shape of the harbour at Holyhead arose from incremental changes as a smaller harbour had been lobbied for by "*the Liverpool interest*", but evidence of heavy use had then demanded an expansion of the harbour.

**Mr Bidder** would not be assuaged, returning to the attack on costs at Portland, and the disparities in cost statements arguing "*and an annual sum of £50,000 disappeared in some unaccountable way, …*".

**Mr Cooper** believed that the breakwater construction at Alderney was near the form espoused by Mr Bidder. He defended St Catherine's, although acknowledged that "*some change of opinion appeared to have occurred as the Government had only authorised the construction of one arm … neglected the other.*" He agreed that that harbour was indeed unfinished, and therefore "*useless for the purposes intended.*"[69]

**Sir Harry Jones** re-stated the need for competitive bidding, and suggested the need for review by leading members of the profession (surely that was the intention of the Commissions). He (again) reiterated that parapet walls should be deleted wherever they were not completely essential.

[68] See discussion in sections 2.5 and 2.8.
[69] Discussed further in Allsop (2020), Appendix C.

**Mr Redman** commented on various locations along the South coast that had been considered for Harbours of Refuge in 1840, 1844, 1857 and 1858.

**Sir John Rennie** discussed the successes of breakwaters at Algers and Marseilles. He noted use of '*beton*' to form large blocks, composed of 2/3 small stone and 1/3 hydraulic lime or pozzolana for blocks of 4-15m$^3$, but noted apparently rather high costs relative to those for rock used at Plymouth.

**Mr Rennie** understood that the Harbours of Refuge Commission would recommend expenditure of about £3,000,000 for harbours of refuge. As this sum "*was so limited, the cheapest plan of construction, combined with efficiency, must necessarily be selected.*" He noted the suggestion of timber frames, but he also noted destruction of the timber cones at Cherbourg, one in as little as 4 days! He then described the extension of the port of Marseille which he had visited in 1857, where the outer breakwater was formed as a rubble mound covered by prismatic beton blocks of 10m$^3$ (probably ~25t, see below) at 1:1 underwater, and 1:2.5 above.

In considering vertical (or battered) walls, Rennie noted heavy overtopping on the seawalls of the South Devon railway (Dawlish?). He also referred to Mr Stevenson's measurements of wave force at Skerryvore, where the maximum reached 6083 lbs/ft$^2$ (292kPa) on 29 March 1829. The highest measurement at Bell Rock was only 3013 lbs/ft$^2$ (144kPa). He concluded however that the subject of wave loads "*... was so complicated that it was almost impossible to lay down a correct theory.*"

**Mr Murray** returned to comparative costs of British and French breakwaters. He gave further details of French rubble mounds and concrete – density for Marseilles beton of $\rho_c$=2600kg/m$^3$. Algers was initially formed entirely of 22t blocks (shape un-stated, but assumed from the diagrams to be rectangular. He discusses the use of hydraulic lime and pozzuolana in making these concrete blocks, describing French experiments on the durability of different mixes.

**Mr Murray** discussed the use of the timber cones in forming the breakwater at Cherbourg, noting particularly the speed of construction. He then contrasted costs at Cherbourg with those for the current works at Dover. The discussion on costs was joined by **Mr Cooper** and **Mr James Brunlees**.

**Mr Brunlees** returned to the (projected?) construction at Blyth, noting the potential for settlement damage as the timber would decay, He referred to a 'wavescreen' being used to give protection to the wall footing. He argued that changes of slope introduced weaknesses. He supported the use of iron in breakwaters, and discussed an open pier sent out to the River Plate.

Noting earlier reference by General Harry Jones to forming an embankment at Dover using local (cliff) material, **Mr W.B. Hays** described works at Port Elliott in South Australia where rock was extracted from a granite headland. The quarried granite sat at a general slope of 1:2 in the sea. He then continued by discussing an iron skirt breakwater in Holdfast Bay, near Adelaide, in ~12m of water with the 'skirt' from 2m above Low Water to 4m below.

**Mr AJ Robertson** suspected that wave forces in shallow water could exceed those in deep water, although he does introduce a confusion "*merely as an oscillation of the water without horizontal movement…*" He moves on however to a discussion on the hydrodynamics of a solitary wave generated in a uniform canal, without at any stage identifying the source of his information, be-it experimental or analytical. He describes a number of differences between waves of translation and oscillation, and presents some (rather suspect looking sinusoidal?) wave equations. He then turned to discuss the Alderney breakwater where the 'first wall constructed with a moderate batter (9" in 1'), but 200ft (60m) was "*... nearly swept away in a storm…*". The new work used dressed stone backed by stone "*rubble set in hydraulic mortar, and pointed with cement…*"[70] In another storm in November 1850, the waves shoaled and broke against the parapet wall "*…with tremendous force.*" Robertson also noted the movement of stones on the foreshore, moving alongshore and cross-shore. He draws

[70] Does he mean Portland Cement?

the conclusion that the mound should be faced by granite. He concluded with "*...he believed the perpendicular wall from the bottom was the true form for breakwaters...*".

**Mr W. Heinke** observed that "*...recent improvements in diving-apparatus had ... enabled the diving bell to be superseded.*" At Dover, "*a diver could now place eight blocks ...*6-7t *... in four hours, whereas a diving bell could only manage four or five*". He understood that currents of 3 mph (1.3m/s) or depths greater than 30ft (9m) gave difficulties to divers.

The discussion concludes by comments from the author. **Mr Scott** countered assertions of similarities with the timber frames, emphasising the differences of the frames at Blyth with Mr Shout's timber carcases, and those used at Genoa and Cherbourg. He emphasised the important distinctions between the forms of waves, and the purpose / role of the parapet. He then presented three lengthy Tables showing cost comparisons for a (notional or Holyhead?) breakwater at a mean depth of 52ft (16m). He particularly included the cost of interest where harbour dues could not be levied until a sufficient length of protection could be assured.

## 11.8 Scott M (1860) on Breakwaters

This paper follows Scott (1858) as the remaining (inner) part of the Blyth breakwater had been completed. Scott remarks that the timber frames at 10' (3m) centres were "*erected with the greatest ease*" despite currents up to 5.6km/hr (1.6m/s). Panels of planking were then attached to the piles. He notes that criticism was made of the strength of this construction, yet he claims that it had survived "*for four winters without injury*", and that the storm of 25 October 1859 was the most severe for many years. With regard to durability, Scott claims that "*whilst at Blyth timber in its natural condition is rapidly destroyed by the worm, the creosoted timber has undergone no change during the past four years*".

He then extends his topic to give advice to engineers, which from this perspective look like statements of the obvious. 1) The site should be of sufficient area and depth, and it should be possible to "*obtain suitable materials for constructing the breakwater*". 2) The direction of entrance(s), whilst mainly "*a nautical question*", may influence siltation. 3) The efficiency of the harbour is in proportion to the "*freedom from undulation in the harbour*". 4) The depth of water must be maintained, "*and silting prevented, as far as possible*".

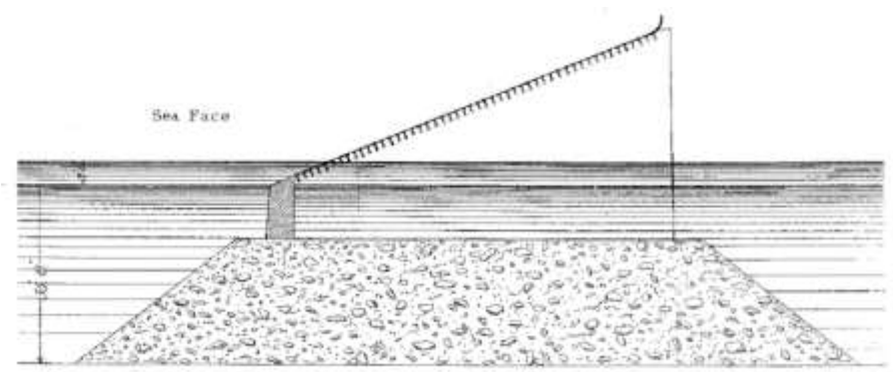


**Figure 11.5 Proposed Table Bay breakwater**
*After Scott (1860)*

Having enumerated these principles, Scott then considers a possible breakwater for Table Bay, South Africa, drawing initially on examples of Plymouth, Alderney, Holyhead, Portland, and Dover. He criticises Plymouth as needing a great quantity of stone, and being expensive to repair. He identifies that the ocean rollers at Table Bay would break violently onto a wall of the form of Alderney. He sees no advantage of burying the wall by further armour as at Holyhead and Portland, doubting that the rock would stay in place, ruining the harbour "*by the large stones being strewed over the anchorage*". Comparing with Dover, he remarks that "*a wave rises against a vertical wall to twice its height", and that the pressure of the water against so high a wall would* *require great width of structure to afford the necessary stability*" – all true! And then he makes his proposal for an open gridiron breakwater, a modification of that originally proposed in Scott (1858), as in Figure 11.5. He anticipates that waves will "*rush up the slope as a confused mass of water*". He also suggests that the breakwater be placed "*at an angle with the direction of the greatest sea*" so that the wave action is dissipated over a larger area. [71]

The discussion was led by **John Scott Russell** who noted that "*the momentum of the ground swell*" at Table Bay "*was not to be destroyed by any structure of small weight*". He also feared that oblique attack would subject the regular counterforts along its length "*an enormous pressure*". These "*great waves of translation*" would require substantial mass to resist them. Whilst preferring a vertical wall of masonry, he reverted to his suggestion of a convex section. **WA Brookes** supported the shallow slope as used at Kingstown Harbour, and Portland, but he noted that the pier at Blyth was constructed on a rock ledge from 2-3m above LW, and that waves break over the *Sow and Pigs* rocks about 20m seaward of the pier.

## 11.9 Stevenson D (1874) Design and Construction of Harbours[72]

**Chapter 1** classifies five shapes of harbour, and gives 2 example breakwater section, one steeply battered and the other with a rising curve to the seaward face. **Chapter 2** considers the influence that the local geology has on the forms of harbour and harbour structures, and what may be learnt from natural rock formations on the relative wave exposure. The levels / depths of muddy beds below Low Water are used to identify wave exposure with particular reference to locations up the Firth of Forth.

**Chapter 3** discusses wave generation, particularly in relation to Stevenson's observations of waves, and his 'square root rule':

$$H = a \sqrt{F}$$

where the wave height is H (often in feet), fetch distance is F (often in nautical miles), and a is a coefficient, depending on the wind strength. Stevenson appears to favour a=1.5 where the wave height is in feet and the fetch in miles, "*sufficient for ordinary gales*" and British coasts. He notes however that Hawksley published a formula in 1861 of similar form, but giving "much greater" wave heights. He notes waves at Wick reaching 40ft (~12.2m) with observations of wave heights at sites of lesser exposure. For Bermuda, he notes the arrival of swell waves well in advance of the main storm.

Stevenson then turns to "wave force" (a general portmanteau term for wave height, momentum, erosive power, or loadings on walls). using $sine^2$ or $sine^3$ reductions for obliquity, although he does note the effect of refraction reducing relative obliquity to the coastline. He notes particularly changes of wave direction in Wick bay hitting the breakwater head normally, but oblique to the breakwater nearer the shoreline. In contrast, in discussing the pier at Inverary on Loch Fyne, Stevenson speculates "*that when a strong gale blows, it successively alters its direction with that of the valleys … to counteract … the winding of the loch*." Contrarily, waves in Loch Linnhe reduce as the water area expands at Lismore Island, and a similar effect occurs on the Clyde.

In a brief discussion on wave period / length, he quotes maximum waves in the Atlantic of 43ft (13.1m) height having wave period 16s and length 559ft (171m), and at Bishop's Rock 20ft (6.1m), period 15s, and length 2000ft (610m).

[71] With hindsight it is clear that he is over-optimistic on the degree of dissipation, and degree of transmission – probably much greater than anticipated. Forces on the toe wall were probably also under-estimated!

[72] This "*contribution to Maritime Engineering*" started as a draft for Encyclopaedia Britannica, then became an independent volume (1st edition) in 1864. In 1874, the expanded and revised 2nd edition, was published by Adam & Charles Black in Edinburgh. It contains a great deal of potentially useful material and experience, but it does suffer from insufficient editing with similar material scattered around multiple chapters. In common with later manuals by Vernon-Harcourt (1885) and Shield (1895) there is significant absence of design formulae, although Stevenson is better than many in trying to give generic formulae for waves. Much material here is however repeated, perhaps from a desire to emphasise key points to the reader. This book has been republished by Cambridge Library Collection in 2011 including the 20 plates, and in-text Figures.

In **Chapters 4**, Stevenson starts by recording a wide range of observations of erosion of rock strata, generally by fracture, and movement of large loose pieces of rock (many tons), much derived from experiences around the Scottish coastline. On the narrower meaning of wave forces, he turns to the breakwater at Wick started in 1863 with 5-10t stone blocks set in hydraulic lime mortar, later Portland cement mortar. In 1868, the rubble foundations were washed down to 15ft below LW (-4.6mLW). Then in 1872, a large monolithic block of cemented rock (7.9m x 13.7m x 3.3m) and immediate foundation weighing in total 1350t was washed away, "*gradually skewed round by successive strokes*"[73].

Stevenson then describes the development and use of his marine dynamometer, see Figure 11.3, measuring wave forces on Skerryvore and Tyree, see Stevenson (1849). His maximum wave pressure measured was 6083lb/ft$^2$, equivalent to 292kPa. He notes that these wave forces varied with elevation relative to water level, also noting many other quantifications of wave loads of order 70-150 lb/ft$^2$ (3.4-7.2Pa). He notes the differences between quasi-static vs impulsive loads, describing further experiments in dropping cannon balls onto the sensing plate of the dynamometer.

Then in **Chapter 5** he discusses conditions that affect wave forces, noting the influence of tidal streams to steepen waves, or to "*act as a breakwater*", wave current refraction bending wave rays away from the harbour or works of interest, but having no beneficial effect at / near slack water when the tidal velocities abate. He continues to discuss "*roosts*" where swell waves are steepened by opposing tidal flows, identifying conditions around the Pentland Firth and elsewhere around Scotland. He links the occurrence of extreme waves at some sites to particular times in the tide cycle, often 1-2 hours before HW.

In discussing depth-limited waves, Stevenson notes that often the largest waves are broken before reaching the site of interest, so no longer the most damaging. He also (*de facto*) notes the effect of the approach bed slope, although doesn't identify slope per se. He cites Scott Russell's comments on depth-limiting wave heights, deriving:

$H= d/2.5$, which might be written $H = 0.4d$

then later he gives guidance on wave crest elevations relative to swl, $\eta$:

$\eta = H/2 + 0.785\ H^2/L$, which might perhaps be written:

$\eta = H\ (0.5 + 0.785\ H.s)$ where the wave steepness is $s = H/L$

At Wick he notes wave crest of about 2/3 wave height and troughs at 1/3.

Notionally, **Chapter 6** addresses harbours in deep water, but Stevenson starts with a discourse on the mechanisms by which vertical or sloping breakwaters dissipate and/or reflect wave action, citing discussions at ICE on the forms of breakwaters between Colonel Harry Jones and Sir Howard Douglas, probably in the discussion to Scott (1858) in section 11.6 above. Stevenson reminds the reader of the differences between 'oscillatory waves' and 'waves of translation', and repeats assertions of the effects of (tidal) currents steepening and breaking waves. He also notes the propensity for damage during construction, i.e. before the works are complete. [74] He also comments on the progression of damage to (new) masonry where the topmost blocks are more easily overturned and removed.

Much of the following material in this chapter should perhaps have been incorporated into Chapters 4 or 5, especially discussion of further uses of his dynamometer on a seawall at Dunbar. The interpretation of those measurements has however sown some confusion by comparing the 'clapotis effect' at a vertical wall in comparison with loads from unobstructed waves acting on a free-standing pile.

[73] This might perhaps suggest impulsive loading, see Wick case study discussed in 6.2 and summarised by Allsop & Bruce (2020)
[74] Of course, this may be because the structure has not acquired full strength, or simply because the design was at fault and would have suffered damage whether complete or not.

In considering breakwaters with long slopes, he repudiates any comparison between Cherbourg and Plymouth, identifying the substantial differences in wave exposure of the two breakwaters. In comparing piers at Dunleary (*sic*) and Kingstown, he reminds the reader of the substantially different depths at the two structures, despite their apparently similar exposures.

A further discussion on effects of obliquity and structure slope introduces some rather complicated (and theoretical) geometrical arguments, then leading to Stevenson disavowing his own advice.

On the resistance of walls, Stevenson cites work by George Rennie giving a friction coefficient between rough blocks of $\mu \approx 0.8$, although he notes his experiments with polished ashlar blocks gave $\mu \approx 0.5$. He then cites pull-out tests on a block from a column of blocks giving $\mu \approx 1.1$,[75]. He then repeats various pull-out tests deriving values of $\mu$ for stone blocks in air and water, showing no systematic difference. He uses the results for polished blocks versus rough hewn to argue against the "*evil of fine workmanship*" noting that effort to dress stones finely may reduce their frictional resistance, perhaps reducing overall stability.

Stevenson then reverts to his previous discussion on the forms of waves (breaking), quoting from Scott Russell's contribution to the ICE discussion (probably to the paper by Scott, 1858, in section 11.6 above). Again he emphasises the dangers from "*waves of translation*" recommending that mounds facing such waves should be 'convex' in profile, steep at the toe with the upper slope becoming gentler towards the water surface, [as adopted for many wave absorbing beaches in hydraulic laboratories], rather than the concave form of e.g. Bremner's pier at Wick, which might be more appropriate for oscillating waves.[76]

Considering a range of breakwaters around UK and French coasts, he compares seaward (and inner) slope angles, although admitting that the data are of little value as not normalised for depth or exposure. He emphasises however that there is no single universal solution, although he notes that structures using "*Portland cement go far to meet the difficulties arising from want of continuity in marine masonry*".

Under 'novel designs', Stevenson then cites vertical (or sloped) wave screens as suggested by Captains Vetch and Calver, and by Scott (1860), although he does appear to display an over-optimistic view of their degrees of wave dissipation.

He completes the chapter with recommendations for harbour capacity, suggesting 5.3 vessels / acre for mixed use, 4 vessels / acre for large merchant vessels, or 3 vessels / acre for a (small) harbour of refuge.

Having nominally discussed deep water harbours, **Chapter 7** turns to discuss harbours in shallower / tidal depths, preferring weight above strength for the breakwaters, as quoted for lighthouse stability. The discussion on lighthouse construction continues including structures at Carr rock, Eddystone, Dhuheartach (now Dubh Artach) (WSW of Iona), and Bell Rock (off Firth of Tay). Back on wave forces, Stevenson then describes further dynamometer measurements on a seawall at Dunbar showing a (relative) distribution of pressures with an enhanced peak pressure at HW (see examples by Hull & Muller, 2002).[77]

More successful were measurements of up-ward acting pressure on a down-ward facing dynamometer mounted at the seawall crest giving a peak pressure of 2352 lbs/ft$^2$ (113kPa), of order 100x greater than the horizontal pressure on the wall about 0.5m lower. He follows this observation by noting that a projecting 'string course' on a wall at Stonehaven "*had to be hewn off*" to prevent occurrence of impulsive loads which had been shaking the masonry above. The consequences of wave impacts are also shown by spray / up-rush giving relative run-up of order $R_u/H \approx 7$. He notes that returning water may then cause substantial down-fall pressures.

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[75] Probably due to the 'jamming' effect seen in the "Orphan Breakwater" failure tests discussed in Chapter 7

[76] There appears to be flaw in this argument because waves in shallower water, where a concave profile might be easiest constructed, are more likely to be 'of translation', whereas oscillatory waves (neither shoaled or broken), will only be found in deeper water.

[77] In discussing the Dunbar measurements, Stevenson reveals (rather obliquely) a problem known to field and/or laboratory experimenters (or to their supervisors!) where the results were not processed at the time, but later found that several years' worth of measurements had been systematically corrupted!

He cites a temporary protection using timber planks devised by James Bremner at Wick to cover a roadway against down-fall loads.

On negative (suction) forces, an inward facing dynamometer on a pile at Dunbar measured pressures 3x greater than the incoming load. He continues by discussing various examples of poor construction where air / water transmission through blockwork construction was significant, leading to remarks on positive or negative air pressures. He cites the outward failure of a door on the Eddystone lighthouse by severe negative (air) pressure differences, and movement of a wall block at Buckie, later secured by a vertical bolt. [78]

Discussing durability of rock, Stevenson notes that timber planking fixed to masonry may be considerably more durable against abrasion than limestone blocks. He presents measurements of abrasion resistance showing the superiority of granite over greenstone. He then shows a table of densities of a wide range of rock types, and of beton [perhaps OPC concrete]. Curiously, he then follows with a table of hydro-static pressures under heights of seawater of 1-100ft (0.3-30m).

Having commented on beach profiles, he gives advice that "*the founding courses of the wall should rise at a very small angle with the beach*". He recommends the use of angular rubble to form foundation so that "*the waves are swallowed up by the interstices*". He cautions however that masonry walls built on / into beaches may be "*deprived of support by the falling water*", noting the presence of scour pools around the angles of large boulders. To protect against such scour, he favours "*a horizontal or nearly horizontal apron … connected with a vertical wall by a quadrant of sufficient radius … causing the alteration in the direction of the wave to take place at that part where the wall is strongest*", although he acknowledges the difficulties in regions of significant tidal range.

He then completes the chapter with a story on the sliding of a stone pier on a clay foundation, moving some 50m horizontally and 12m vertically. He also reprises the suggested construction of a lighthouse on the Godwin sands, again preferring the use of weight over mechanical fixture.

**Chapter 8** advises on harbour layout, for which precursors are "*numerous and accurate soundings so as to give a correct representation of the bottom*". On layout, Stevenson highlights the danger of requiring vessels to cross waves obliquely approaching of in the entrance. He recommends facing the entrance to coincide with the heaviest seas, although in his Fig 26 he illustrates a neat solution where an outer breakwater projecting past the entrance approximately normal to the waves is used to turn wave directions in its lee towards a narrow entrance tucked into the lee of the outer pier, thus perpendicular to the original wave direction. This might ease navigability, but could actually assist wave penetration.

On entrance widths, he cites data from ~25 harbours showing widths of 30m to 300m. Beyond the entrance, he recommends that ships need a "*good loose or point of departure*", a clear water area within which to establish speed and direction having cleared the harbour entrance. He lists the causes of 266 wrecks including 5% wrecked on piers or bars through insufficiency of tugs; a further 22% stranded or struck on rocks, bars or piers whilst entering or leaving port; and 45% stranded on rocks or sand[79]. He deduces however that the mariner encounters "*his greatest perils when he is nearest to his port of destination or departure.*"

On the orientation of piers, he strongly cautions against concave outline (in plan) or abrupt re-entrant corners, preferring to suffer the complication of changes of foundation level rather than follow a contour if that leads to adverse plan-shape.

Having lamented the absence of any (standard) formulae / method to predict (diffracted) wave heights within harbours, he then develops methods of his own, developed with observations by his son (Robert Louis Stevenson) at Pultney Town (south side of Wick) giving a reduction factor for wave height:

[78] This block movement is extremely similar to that described by Muller *et al* (2002) and Marth *et al* (2004).
[79] I assume some of these are remote from any port

$K_{diff} = 1 - (0.06. \sqrt{\alpha})$ where $\alpha$ is the angle of deflection (in degrees)

This implies a wave height reduction of $K_{diff}$ =0.8 at 15° off the wave direction, and $K_{diff}$ =0.66 at 45°. He notes in passing that he ran some experiments in a brewer's cooling tank about 0.1-0.15m deep, acting as a miniature wave basin, from which he concluded that (diffracted) wave heights reduce directly as the distance travelled, and as the square root of the angle of deflection. He then develops a formula for a 'close' harbour estimating wave height at a radius *D* from the harbour entrance, for a local harbour width of *B*, and a harbour entrance width *b*. He compares predictions using this method with observations at Kingstown, Sunderland, Macduff, Fisherrow, and Buckie.

Stevenson cites a number of methods to reduce waves in a harbour including: interference chambers of '*clair voie*'; side channels to the entrance as used at Hartlepool; and spending beaches of 1:3 to 1:4.

He also identifies a number of sites where the wave action might normally be tolerated without the shelter of a breakwater with fetches up to 7 miles, although his recommendation is to limit that to a maximum fetch of 5 miles.

**Chapter 9** covers docks, locks and slips. If the harbour shape cannot itself reduce (extreme) wave conditions sufficiently for the required harbour operations, then a gated dock of tidal basin may be needed using gates, a caisson, or stop-logs to exclude wave action. A locked basin will allow vessels to remain alongside to be worked, whereas in a tidal basin, vessels will often ground with the potential for damage that might entail. Stevenson lists dimensions for docks around English, Welsh and Scottish coastlines, together with advice from France and the Low Countries.

He describes the purpose / use of graving docks for ship repair, recommending that the dock be built with stepped sides to facilitate propping the vessels, and for carpenters to work off, rather than sloping sides. As alternatives, he notes floating docks, or hydraulic lifts, part-tide gridirons, or (steam powered) slips with sliding boat carriages. But of all of these, Stevenson prefers the graving docks, enumerating six reasons for his preference. He then concludes by discussing dock gates.

**Chapter 10** discusses materials and implements. Citing texts by Horace, and a construction contract (in Latin) between the Abbots and Burgesses of Arbroath, he suggests that timber has long been used for "chests filled with rubble stones" to form harbour structures[80]. The main problem for construction in timber is its durability against "*marine insects*", often *Teredo navalis* or *Limnoria terebrans*. These are "*not … of recent appearance on our shores … upwards of 300 years ago …*". He compares the resistance to attack of timber types with experience of trials at Bell Rock of 50+ different timber types / sources. He cautions that greenheart timber as then being imported may not be as resistant as previously expected. He also notes that use of creosote may not give protection if the timber is cut or drilled after creosote is applied.

On pile strengths, he cites formulae by Dr WJM Rankine. Somewhat at variance with his previous preference for "*weight above mechanical fixture*" he describes the advantages of timber frames for load transfer as opposed to masonry. Then perhaps to balance the argument, he describes destruction of stone strata or masonry blocks by molluscs, including damage by them to shale beds at Kirkcaldy causing the outer parts of harbour walls to settle more than the backing.

As well as durability tests on timber (see above), 25 different types or combinations of 'iron' had been trialled on Bell Rock.[81] Galvanised specimens resisted corrosion for only 3-4 years more than the iron samples. Wrought iron corroded at a rate of 1/8 of cast iron, although the rate is substantially affected by the quality

[80] See Routh (1912) and section 2.3 for the example at Tangier.

[81] Stevenson appears to use the term 'iron' rather generically, with 'wrought' and 'cast' sometimes applied, but it is unclear whether 'iron' might sometimes mean 'steel' or similar.

(permeability / density) of the specimen. Having cited many examples of decay of 'iron' elements, Stevenson cautions against "*the indiscriminate employment of iron in marine works*".[82]

On use of masonry, Stevenson suggests that (some) requirements of marine masonry are "*nearly the opposite of those for land architecture*", arguing that the type of interlock leading to bridging over a removed block are not attractive. He also recommends that the backing material should "*be carefully set, and regularly bonded with the face-work*." On permeability, he discusses the potential for trapping and/or venting air from beneath a roadway. Rubble filling for breakwater walls should be "*generally of much larger size than for ordinary commercial piers."*[83] He suggests that the void ratio of the fill might be between 0.11 to 0.33.

Discussing the use of "*stones on edge instead of on their beds*" as in many old Scottish breakwaters, seawalls and quays, he considers that this may give "greatly superior strength", although "*the advantage is most conspicuous where the materials are thin*." He cautions however against use for strongly oblique attack that might "*strike the masonry on the overhanging side*".

For conventional blockwork, additional stability may be given by dovetailing, or "*treenailing*" with timber pins in beds or joints, but the complication [and expense?] may only be justified "*where the loss of a single block is certain to occasion great inconvenience and delay*".

"*Beton*" [OPC concrete?] "*is now very commonly used in this country*". [Again, I assume in 1865.] Stevenson recommends a specification for cement by Sir John Hawkshaw. He suggests concrete of: 1 part OPC; 4 parts of sand; 5 of shingle, although he later describes without comment concrete blocks of 1 part OPC to 7 of stone. On the use of concrete in marine construction, he cites:

- 50t blocks cast in stages, lifted by pairs of lighter;
- concrete deposited underwater from bottom-opening boxes;
- concrete laid underwater in large hessian(tarpaulin) bags;
- concrete poured into timber formework using thin iron key strips to leak-proof the timber planks.

Stevenson quotes descriptions of concrete making by Messent, Mr Deas of Glasgow, BB Stoney of works at Dublin, see also Stoney (1874). He also appears (over-) excited by the very recent (during proof-reading of the book) discussions on "carbonite cement" being promoted by Dr George Hand Smith of New York, a metallurgist. [84]

He then describes placement of rubble foundations or mounds from timber staging at Portland, Plymouth (Mill Bay), and Holyhead. At Wick (Pultneytown) harbour, he notes the high frequency with which greenheart timber "*piles were invariably broken by the waves at about the level of high water*".[85]

**Chapter 11** is primarily concerned with the use of tidal or fresh water to flush out sediments, although it does also discuss saline wedges travelling upstream against out-ward flowing fresh water.

His last section, **Chapter 12**, is titled "*miscellaneous subjects*", so draws together a wide range of topics not (or thinly) covered in the previous chapters. This review will primarily select for comment those topics of relevance to breakwaters.

A major issue in harbour design is siltation, often described here as '*shoaling*'. Stevenson discusses siltation of enclosed harbours, and possible mechanisms to scour out sediments by releasing concentrated flows. An example at Sunderland discharges 444,000ft$^3$ in 15 minutes, giving an outward velocity of just over 4 miles per hour. He quotes work by Sir John Leslie and Mr T Login in identifying threshold velocities for movement of different sediments. Stevenson then notes that steam-powered dredging was "*in frequent use both for*

[82] It appears from the general discussion on corrosion suggests that Stevenson expected that corrosion was a direct chemical reaction applying to submerged iron, ignoring the increased effects inter- or supra-tidally, and of the abrasion effects of sediment removing corrosion products thus accelerating the losses.
[83] Assumed to mean of lower wave exposure.]
[84] The text on carbonite is heavy on claims, but substantially lighter on key details.
[85] Does this provide supporting evidence for the frequency / strength of impulsive breaking by waves shoaling over the breakwater mound?

*formation and preservation of harbours*" [86]. The steam dredger used on the Tyne had a 50HP engine, cost about £20,000[87], and lifted at best about 450-500 tons in an hour when uninterrupted. He continues with comment on other types of dredger, and use at other locations. The dredger working at Lough Foyle [short period waves] could still work to waves of 2.5ft (0.8m) although at other locations the limit might be 0.6m.

As a siltation prevention measure, Stevenson notes planting of Pinus maritime and bent grass to stabilise sand dunes and avoid sand being blown into the shipping channel at Mullaghumore harbour, Co. Sligo. Having discussed ways to flush out or dredge entrance channels, he does however remind the reader that this can allow greater wave penetration into the harbour, noting such problems at Lybster, Dunbar, Cockenzie and Sunderland.

On the positive side, he does note that the (shipping) capacity of a channel does increase by the cube of the depth, so a relatively small degree of dredging may allow larger ships (with cargoes of greater value) to use the harbour more often. To illustrate, he then presents formulae and tables relating vessel tonnages to required draughts, continuing with a discussion on under-keel clearances, termed "*scend*".

A further section discusses lighthouses, referring the reader to books on the subject by Alan Stevenson and Thomas Stevenson. Amongst further topics are:

- Suspension piers [this was before the Chain Pier at Brighton had failed!];
- Timber ponds;
- Reasons for two entrances – he notes the possibility of entering the then Peterhead Harbour from either north or south;
- Screw piles and screw moorings;
- Hydraulic operation of gates, bridges, and sluices;
- Timber fenders;
- Coal loading and coal staithes;
- Floating landing platforms formed by multiple iron pontoons (30 on the Mersey);
- Capstans
- Steam cranes – examples given of 50-70 ton capacity in Glasgow and Greenock;
- Piers of cast / malleable iron;
- Funding by government or local funds.

## 11.10 Stoney (1874) on use of large concrete blocks

Stoney noted the "*…great cost of temporary works…*". "*In numerous cases … when foundations are laid within cofferdams at considerable depths … the cost of temporary works is found to equal or even to exceed that of the permanent structure.*" His solution was to use "*artificial blocks of such magnitude*" to bring the structure above water level. He had used these on a regulating layer to form quay walls at Dublin placed by floating shears (derrick or crane) with the foundation trimmed using a diving bell, both constructed in 1864-65. Speed of placement similar to that needed for a block of 1/10 its volume. The blocks, 27'H x 21'4"W (at base) x 12' L, volume nearly 5000ft$^3$ (142m$^3$) and 350t. Each block has a rectangular groove 36" x 18", making a 3ft square dowel (0.9m), filled by in-situ concrete, although "*no doubt, the blocks would act very satisfactorily if the dowels were omitted*". [88]

These large blocks were faced by stones set in 4:1 sand to Portland cement. Then the mass was formed using (significant quantities of?) stones 2t and down, set into concrete formed by 1 part (volume) of Portland cement and 7 parts limestone ballast (sand and gravel dredged from the harbour). Once set, the dowel grooves between adjoining blocks are plugged by concrete dowels. The average joint width over 100m of wall construction was about 15mm, between blocks 3.7m long in the line of the wall. In discussing costs, he notes that OPC is about 20% more expensive in Dublin than in London, presumably due to costs of supply.

[86] So both capital and maintenance dredging.
[87] Perhaps £2.4 million at current prices.
[88] This rather implies that he made no calculations of the disrupting loading.

The placing vessel (barge) shown here in Figure 11.6 was essentially rectangular (39.6mL x 14.6mW x 4.3mD) with a semi-circular bow. A balancing tank at the stern counter-balance the load on the shears projecting over the bow. A system of timber booms or props bear on the lifted block to reduce any chance of taking any significant list and over-turning. He discusses (at some length) the chains used to suspend the large concrete blocks, and the balancing tank. All of the lifting is operated by "*... an ordinary horizontal high-pressure engine of 14 nominal HP...*" operated by a boiler producing steam at 45psi (13kPa). This implies that such engines were relatively easily obtained when the lifting shears were constructed in 1864.

Stoney continues by describing the 80t diving bell used in the works at Dublin, 20' square at the bottom, 16' square at the top (so 6.1m square tapering to 4.9m square) and 6.5'high inside (2m). To the top of the 'bell' is attached a tube or 'funnel' around 0.9m diameter and some 13.5m high. The upper end of this 'funnel' forms an airlock to maintain pressure within the diving bell whilst permitting transfer of operators or supervisors without lifting the bell. Material excavated from the seabed is deposited into trays within the bell, to be dumped later once the bell has been lifted and moved to above a suitable place on the seabed, unloading the trays by releasing the latches. Compressed air is delivered to the bell having been cooled, pumped by a small steam engine. A further steam engine drives the lifting arrangements on the barge hull constructed by Harland & Wolf, but fitted out by the bell manufacturer, Grendon & Co. of Drogheda.

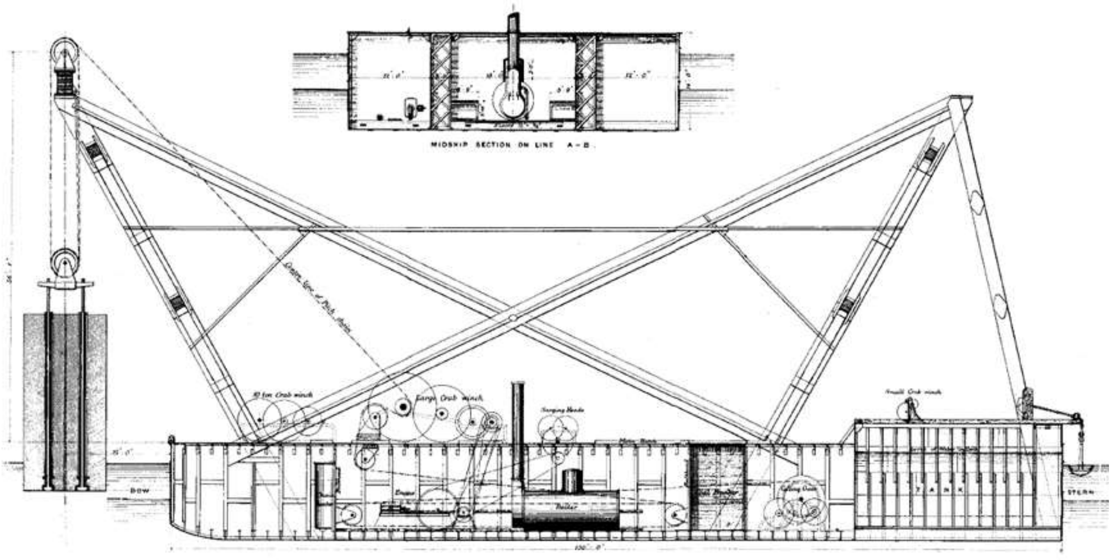


**Figure 11.6 Block placing barge**
*After Stoney (1874)*

He discusses methods of mixing dry components of concrete to achieve consistent mixing, and the advantage of machine mixers driven by a 3HP (steam) engine in maintaining correct and even water content.

Stoney then completes his paper by discussing the use of these very large blocks to form breakwaters. He starts by categorising the main configurations in use, with examples. He then notes that the two principal types in use "*… contain the elements of destruction within themselves…*", and later "*... the destruction of the superstructure of the breakwater is only a question of time.*" He then enumerates mechanisms for failure, particularly those involving movement and abrasion / attrition of foundation material. In contrast the deeply-founded vertical wall avoids such foundation problems, although construction costs may be greater and progress slow, although both may be ameliorated by use of the form of construction described in the paper.

## 11.11 Discussion to paper 1376, pp 356-380

**Mr Abernethy** congratulated the author on the pioneering use of large blocks and agreed with the suggestions to avoid the problems experienced at Alderney.

**Mr Parkes** compared Stoney's approach to mixing concrete in troughs to his own use of Mr Messent's mixers at Kurrachee which "*... produced almost perfect concrete.*" He did not however accept the need for such large blocks in breakwater construction. At Kurrachee, the sloping blocks of 27t each were moved and placed by land-based plant of cost £6,300 in contrast to Mr Stoney's marine plant of cost around £18,800. The use of the '*slicework*' system also simplified the foundation requirement, reducing excavation volumes. He also refers to use of a steam dredger at Kurrachee used to excavate a trench for the blocks, then levelled by divers. He then compared costs of manufacture and placement between Dublin and Kurrachee.

**Mr C.B. Lane** considered Stoney's invention(s) "*…a stride in the direction of economical construction of engineering works in deep water*." He then wondered how they might be applied to forming bridge piers.

**Mr Vernon-Harcourt** was much interested and clearly approved that "*... great solidity and durability had been attained at a comparatively moderate cost.*" But he felt that the author was too optimistic on the number of days lost in exposed situations.[89] Vernon-Harcourt particularly noted the weather susceptibility of levelling of foundations for the lowest blocks. Diving bells had been used at Alderney, but the fast currents "*drove the men to the sides*" and the bell was replaced by using helmet divers. He also noted the expense of the plant, "*…large expenditure of cement…*", and need for "*shelter for the barges*".

Vernon-Harcourt further discusses movement and abrasion / attrition of stone at Alderney, although (misguidedly?) refers to "the equally hard native Mannez stone" [90]. Indeed, he does note the transport of "*small rounded stones ... round Grosnez Head into the adjoining bay ...*".

**Mr Stoney** gave further details of (the rates of) recent construction, noting however that these were obtained in "*... a sheltered harbour*." He had recently placed 140t blocks, and suggested that the large blocks described in the paper as 350t might have been closer to 400t. It appears that the Ballast Board may have become nervous and requested that he obtain "*... a second opinion on work of such a novel character ...*" The report by Mr Bateman "*... was too flattering for him to repeat; but it was a great encouragement ...*".

**Mr Parkes** answered a question on the sliced blockwork at Kurrachee explaining that the original intention had been for vertical stacks, but that these would not have been stable. The stacks had therefore been set back at a 4:1 batter. He recommended that "*... the great object ... was that each block would rest upon a single block below, and have no break of joint.*"

**Sir John Hawkshaw** (ICE Past President) bore testimony to the success of "*... works of a novel and interesting character*." He cautioned that it was not necessarily possible to generalise "*… from a particular structure ... Engineering was not so simple....*" he cited Holyhead as a site where it should not be applied "*... there was a mountain close at hand to obtain stone from at little cost.*". He clearly found favour in Mr Parkes' suggestion of sliced blockwork, but then contrasted costs of rubble mounds with tall blockwork walls. He asserted that lower capital costs and small annual maintenance costs might be preferred to high capital costs.

**Sir John Coode** supported Sir John Hawkshaw's comments, both on the methods used, but also on the difficulties in generalising to other sites. There was a brief side-discussion on stone density – Mr Stoney calculated that the stone he had used at Dublin gave 14ft$^3$/t (2.53t/m$^3$). Sir John had made concrete blocks using Silurian (lime)stone of density 15.33ft$^3$/t (2.77t/m$^3$). He questioned the implications of 'novelty', asserting that similar methods had been used to cast and lift concrete blocks, indeed that use of large concrete blocks could be credited to the French. He believed that the credit for inclined blockwork should be given to Major Askwith

[89] Here we see the different perspective arising from construction experience at Alderney.
[90] Which we now know to be abraded to be the main source of sand and cobbles on Platte Saline and Grosnez Beach, see Allsop et al, 1991.

of the Royal Engineers who had proposed its use for Dover. He commented on the allocation of plant costs, noting from his own experience that "*… no one knew how to use special plant when the particular work was finished.*" On utilisation, he felt that 200 days / year was extremely optimistic, he suggested 40-50 days/year as more realistic on many (more exposed) sites.

**Dr Pole** supported Sir John's ascription of credit for the use of large concrete blocks to the French.

**Mr Bateman** (ICE Vice President) confirmed his positive analysis of the methods used at Dublin. In relation to the discussion, he noted substantial differences between the French use of blocks up to 16t (made of large gravel and hydraulic lime) and Mr Stoney's blocks of up to 400t.

**Mr W Dyce-Cay** thought that use of the block-placing barge would not be safe for breakwaters where swell might well induce significant rolling. He therefore thought that it would be very expensive to make and carry such large blocks. He then described his own use of 'liquid concrete in bags' as used at Aberdeen.

**Mr J L Thornycroft** suggested that the barge carrying the lifting shears could be redesigned to reduce need for the balancing weight, reducing construction costs.

**Mr Redman** categorised the construction on the banks of the Liffey as "*river engineering*". He also doubted if the construction approach used at *"...the head of the Arabian Sea for a long and gradually shelving foreshore…*" could be applied to Alderney "*...with its rocky bed 20 fathoms deep* (37m)*, a precipitous shore, and an offing* (exposure?) *across the Atlantic.*" "*No analogy existed between the two cases.*" He speculated however that slice blockwork as at Kurrachee might be extended if the blocks were to be of the full width of the wall. In passing, he referred to a Royal Commission on "*unseaworthy ships*" suggesting that it might lead the public to see the need for "*... greater protection of seaworthy ships*".

**Mr A T Andrews** had made large concrete blocks around 20 years previously with around a fourth consisting of Kentish ragstone. At Cape of Good Hope, he had similarly used large stone of density 12.5ft$^3$/t (2.26t/m$^3$) in concrete blocks for dock walls of 10-12t, and of 70t to protect the outer end of a breakwater.

**Mr J N Douglas** questioned whether there were "*... reliable information as to the maximum weight of blocks of stone capable of being moved by the sea… a matter of great importance in the construction of breakwaters.*" Based on experience of wave action around Land's End and the Scilly Islands, he speculated that the safe limit might well be 300t on "*... the most exposed part of the coast.*" Similarly, he concurred that the suggestion of 150 days/year utilisation of floating plant was extremely optimistic, indeed he suggested 30-40 days/year would be more realistic at many (exposed) sites. He noted that blocks of 150-200t were being used at Brest when he visited in 1868.

**Mr Stoney** answered various points made previously in the discussion, particularly on the first use of large concrete blocks, and difficulties in mooring the placement barge. He became aware of Sir John Coode's diving bell through a book published in 1868, by which time his had already been constructed. But both had been anticipated by a French "*... patent in 1858 for a similar contrivance…*" In considering the cost (and further use) of plant, he emphasised other uses for the blockwork wharf and the diving bell. He considered the use of sliced blockwork, but cautioned that the construction needed to be wide enough to accommodate the gauge of rails needed for a 'Titan' crane. He noted the cost of Portland cement in London was 45-55s per ton, but that shipping cement to India would substantially increase its cost. On the topic of sliced blockwork, he noted that the construction at Kurachee had accumulated a 9" (0.2m) cumulative "*fall-back*"[91] along the [unstated] length of construction "*... whatever their cause …not calculated to promote the longevity of the breakwater.*"

**Mr R Gervase Elwes** was particularly impressed by the concrete mixing apparatus at Kurrachee, being "*... revolving boxes … a peculiar angular shape … sides inclined to the axis of rotation. They therefore turned*

[91] I assume at the top of the blocks.

*their contents over completely at each revolution … These combined movements effected a more thorough and rapid mixing, with less expenditure of power …*".

In the final contribution to the discussion, **Lieut.-Colonel Playfair** quoted from his note to the Board of Trade from November 1869 in which he " … *was in the act of … describing the splendid harbour recently completed at Oran … received a dispatch … containing the particulars of a frightful storm which … utterly destroyed all the great works which had been accomplished … speedily scattered the immense blocks of concrete … leaving not one standing on another.*"

## 11.12 Vernon-Harcourt (1885)[92]

### The manual

**Chapter 1** (Preliminary considerations) describes types of harbours, and discusses wind.

**Chapter 2** on Waves cites examples of wave observations, and gives qualitative discussions on wave breaking; and on wave prediction formulae by Hawksley and by Stevenson. Vernon-Harcourt describes and illustrates Thomas Stevenson's "Marine dynamometer"[see also Stevenson T (1849) reviewed above] to measure wave forces (p27). Examples of wave pressures up to 6100 lb/ft$^2$ (292kPa) at Skerryvore and a beacon failure at Petit-Charpentier rocks, back-calculated at 6140 lb/ft$^2$ (294kPa). Notes structural failure of 1350t monolith at Wick in Dec. 1872.

**Chapter 3** describes Tides; and **Chapter 4** forms / configurations of Harbours. Stevenson's formula for wave height reduction within harbours is presented.

**Chapter 5** is a short summary on the generality of rubble mound breakwaters and jetties (pp90-103). The early part is spent discussing definitions of jetties of various types and compositions, piers, breakwaters, wave-breakers, wave-screens, and floating breakwaters – which he mostly dismisses. On classifying breakwaters, he distinguishes mainly between mounds; mounds with superstructures; and upright walls.

**Chapter 6** covers mound breakwaters with superstructure, classifying mounds where the 'wall' is founded at LW (or above) vs. below LW. Vernon-Harcourt rehearses advantages and disadvantages of adding the superstructure, noting the need to resist impulsive wave loads. Examples of superstructures founded at LW are: Cherbourg (requiring rock feeding annually); Portland and Holyhead (notably large blocks used to raise the mound to HW or above; St Catherine's; Marseilles; Alderney; and Tynemouth.

Discussing the mortar for the lower courses, Vernon-Harcourt notes the previous use of Medina or Roman cements, now (1884) superseded by Portland cement (used with one to two parts sand). On superstructures, he notes 'high thin' walls on example French breakwaters, and two walls with rubble infill at Holyhead, Portland, Alderney and St Catherine's.

Vernon-Harcourt discusses the level to which the wall foundation should be taken, noting examples of structures where damage has occurred, and others judged to be relatively stable. He does identify that high foundation structures (where the seaward toe of the wall element is set close to or above LW) can survive in sheltered locations, *e.g.* St Catherine's, but that at Alderney and Odessa (Black Sea), and at the outer end Tynemouth, the mounds were / should be kept at least 20ft (6.1m) below LW. He favours the seaward mound being continued upwards above the foundation level to give protection / support to the foundation blocks of the wall. Somewhat surprisingly, there is relatively little mention of the use of divers until he discusses (briefly) the placement of granite ashlars on the seaward face of Alderney, and concrete blocks on the harbour-side.

92 This is a major treatise on "Harbours and Docks" with examples from around the world, in two parts: Chapters 1-18 on Harbours (including Breakwaters); Chapters 19-28 on Docks. A separate volume gives Plates to illustrate the text.

**Cranes and methods of moving / setting blocks**

A balance crane was built at Alderney to place piles, and then for lowering stones for repair work (perhaps 1850-60). Termed a "Samson" it used a balance weight at one end and extended beam carrying a lifting "crab" (sideways sliding carriage). Blocks up to 4t could be handled at 30ft (9.14m) overhang. The (later) implication was that this was not (steam) powered, so relying on human power.

A steam-powered "Titan" designed by Mr Parkes could lift blocks of 27t at an overhang of 26.5ft (8.1m). Similar cranes were used at Ymuiden (6-12t blocks), Columbo (33t blocks), and at Mormugao (37t), where an extended jib version lifter 20t wave-breaker blocks. A revolving steam crane to lift 15t blocks was designed by Sir John Coode, and built by Stothert & Pitt (see Andrews & Burroughs, 2011). Vernon-Harcourt notes that Titan cranes at Ymuiden and Madras had been washed away / destroyed.

A "Goliath" crane designed by Mr Philip John Messent lifted 40t at 75ft (22.9m) overhang on Tynemouth North Breakwater (probably shortly after 1862).

**Slice- or sloping-blockwork**

He discusses various uses of sloping blockwork by Parkes, Walker and others with inclinations from 76°, 70°, and 48°. Failures at the crest were noted, against which some designers used iron cramps, held by oak wedges, joggle bags (mortar filled hessian bags), and dowel bars.

Vernon-Harcourt concludes Chapter 6: '*... the best appears to be a superstructure founded some 20 ft (6.1m) below low water upon a simple rubble base, and formed by large concrete blocks laid by over-hanging cranes upon the sloping block principle, securely connected vertically and transversely, and capped with concrete-in-mass after settlement has ceased'*.

**Chapter 7** covers Upright-wall breakwaters, taken to be defined as in Chapter 6 where the toe of the wall is (generally) set below LW. Often used as a mooring pier / quay, such walls must offer no significant underwater projection, so any foundation mound must be small and project very little from the wall. The greatest depth achieved for such a wall (by 1884 when the book was completed) was Dover Pier out to 40ft (12.2m).

Levelling the seabed to receive blocks generally requires excavation / filling by helmeted divers and/or use of diving bells. But at Aberdeen (1874-79), William Dyce Cay used bags of liquid concrete (5-16t) to conform to the rock, later extended to 50t or 100t bags for the outer extension of the North Breakwater. Vernon-Harcourt and W Dyce Cay clearly had some correspondence on the placing of these large bags, and a diagram shows a section through the placement hopper barge, and an Appendix by Cay gives further details, including costs. Each foundation bag occupied around 800 cubic feet (22.6m$^3$) when laid out within the specially built hopper barge, and received 50t of liquid concrete (1pt Portland cement, 3 of sand and 4 of gravel). The barge was towed from the filling quay to the selected location. Once moored and winched into position, the bottom doors were opened allowing the bag to fall to the seabed. Multiple layers of bags were used, longitudinal for the first two courses; transverse for the upper courses. Generally 6-7 bags were placed per day with a maximum of 9. The same barge was used at Fraserburgh 1875-1882, see Paxton & Shipway (2007a). A larger barge was used at Newhaven in 1884 to place 104t bags (47.1m$^3$).

Side reference to a breakwater in the Adriatic at Fiume in 22m depth where major sections were cast in-situ in 1150t monoliths cast in 9m long 'frames' leaving 6m gaps to be filled in later. He mentions placement of concrete into these forms by canvas tube, skips with an opening base.

Rosslare harbour near Wexford used piers formed by concrete in-mass deposited up to low water by opening skips into timber frames lined by canvas. Each pier section weight about 220t. Timber frames lined by canvas were also used at Buckie (1877-78), where the concrete was tipped directly from 'ballast wagons'.

Writing in 1884, Vernon-Harcourt describes quay and wall construction by Mr Stoney at Dublin using 350t blocks, cast on shore and lifted directly into position by floating 'shears'. Vernon-Harcourt notes that Stoney has proposed this method for extending the breakwaters, but notes that the method '*has not hitherto been applied … with the exception of a beacon … in Dublin Bay*'.

Vernon-Harcourt compares the advantages of these different approaches. Overhanging cranes (perhaps Titans or Goliaths) can only be used one at a time at the end of the advancing construction, but construction with sloping blocks is rapid, and such cranes can be withdrawn to shelter in advance of a storm. He notes that placement of sloping blocks by Titan crane had been expected to be comparatively slow, but that progress at Kurrachee was faster than expected. He notes the need to connect the upper courses together, capping with solid concrete.

He notes that '*barges cannot deposit concrete bags with sufficient accuracy in rough weather*', but further notes that Aberdeen North Pier advanced by 305m in two years. The advantages of concrete bags are in levelling an uneven bottom, and in ensuring (relatively) monolithic construction. Debating the advantages of such bags over the use of formwork, he favours the former for large projects, but notes that concrete in frames requires '*little plant*' and forms a smooth monolithic mass, so to be preferred where '*foundations are firm*'.

Use of (very) large blocks '*has yet to be tested in exposed situations*', and may only be possible on limited working days, but may possess '*great stability without … frequent maintenance*'.

On use of concrete, Vernon-Harcourt notes the first use by M. Poirel at Algiers in 1834, gradually extended since. He discusses this further in Chapter 11, giving block sizes of 19 and 20 $m^3$., but suggesting that they used hydraulic lime and/or pozzolana rather than OPC.

**Chapter 8** discusses Jetty Harbours with parallel breakwaters / jetties, primarily their configurations relative to sediment movement. **Chapter 9** covers harbours with converging jetties, particularly Dublin, Aberdeen and Sunderland. At Dublin, V-H notes construction of the Great South Wall completed in 1796, length unstated, but implied 3.33 miles, 5.4km. Vernon-Harcourt discusses development of protection to Aberdeen harbour by breakwater lengths by Smeaton, Telford and Cay, but again mostly concerned with the issue of siltation.

**Chapter 10** discusses (mostly the layout of) harbours formed by rubble mounds, particularly Plymouth and Portland, and **Chapter 11** those formed also using concrete blocks. At the former, construction started in 1812 to Rennie's design. Stone from local quarries was run in rail wagons onto special vessels capable of holding four rows of wagons (80t), discharge from a tilting platform at the stern. Having seen damage to the original 1:3 slope, the slope was slackened to 1:5, later paved by granite pitching. Even so, maintenance by regular deposits of stone were still being required (in 1884). **Chapter 12** then describes Mediterranean harbours with sorted rubble mounds, and slight superstructures, most nearly approaching present rubble mound breakwaters.

**Chapter 13** describes composite wall breakwaters with the wall (superstructure) founded at low water, including Cherbourg, Holyhead, Portland and St Catherine's. The latter (see also William Davies' article on the "harbours that failed", and Allsop (2020) in Appendix C) was commenced in 1847, intended to be one of two converging breakwaters and completed in 1855 with the northern breakwater at 610m length. The St Catherine's Breakwater was formed as a mound carried just above LW with the superstructure founded at LW of harbour and sea walls filled by loose rubble. These walls were formed without mortar up to HW, only mortared above HWOST. Work on the harbour was discontinued in 1856 with the southern breakwater having been started in 1849. Repair work on St Catherine's Breakwater is discussed by Hold (2009).

**Chapter 14** describes selected French and Italian composite wall breakwaters with the wall (superstructure) founded at low water, which use both rock and concrete blocks. **Chapter 15** describes composite wall breakwaters with the wall (superstructure) founded below low water, particularly Alderney and Tynemouth. Devoting some 13 pages here to Alderney, Vernon-Harcourt notes the erroneous use of the term harbour of refuge, asserting that Alderney and St Catherine's were conceived for military purposes. He also notes the changes of breakwater section as the work progressed, and "*the final abandonment of the works in an incomplete state, and the discontinuance of ... maintenance for the outer portion*".

The Alderney breakwater mound was formed by hard (?) sandstone (Mannez quarry) with two dry walls with rubble fill forming the superstructure. The walls were (initially) founded at MLWS. The quay level was set at 1.8m above MHWS, thus 7m above the wall foundation. On the seaward wall (battered side, a promenade wall protects the quay, surmounted along part of its length by a small (1.25m high) parapet wall.

This configuration was then adjusted in 1849 at some 125m from the root where the level of the mound was lowered to 3.7m below MLWS, the seawall batter was reduced to 0.5H:1V above LW and 0.33H:1V below. Up to 125m, the mound had been deposited by wagons, but the lower mound required hopper barges (taking loads of 60-140t) towed by steam tugs. The wall blocks were now set in cement (cement mortar?). This section was used from 125m out to 823m from shore.

In 1856 a considerable extension of the breakwater length was decided. For the further lengths, the rubble hearting between the walls was replaced by concreted hearting, the seaward face batter was reduced to 0.25H:1V and the superstructure width decreased by 5ft (1.6m). This section continued in a straight line to 884m, then curved outwards to 1042m from shore, then continued north-east to its end at 1426m (4680ft) from shore.

The mound (average 265,000t of stone per year) was placed from barges (60-140t capacity) towed by steam tugs. Superstructure built from staging. Wall blocks were lowered from the staging and placed by divers on levelled foundation. Spaces between walls were filled by rubble and sand. The final (?) termination built in 1864 was in 133ft of water (41m below LW), with the superstructure founded 7.3m below LW.

Vernon-Harcourt then describes the processes and effects of wave action on the breakwater, particularly the impulsive breaking onto the wall, and overtopping, followed by draw-down moving mound stones away from the wall foundation. He estimates that an average of 50,400t of stone was required each year (1864-1872). He judges that the foundation of the superstructure should have been taken down to 7.6m below LW, 4.9m not being sufficiently low. He discusses breaches in the superstructure in 1865, where face stones were pulled out at about LW, the breach then extending through the wall and venting through the quayside. In summary, Vernon-Harcourt judges that the superstructure foundation should have been at a lower level and taken fully across the full width of the superstructure; the batter should have been reduced to give greater 'clamping' force on blocks at low level; and the upper promenade wall reduced or removed to reduce loads on the seaward face.

The costs of damage mounted, so government commissioned a report from Hawkshaw (Sir J) and Clarke (Sir A) in 1870. They recommended that the promenade wall be lowered, and the rubble foundation mound be armoured by rubble or concrete blocks along its seaward face. The government did not consider that the expenditure was justified and discontinued maintenance of the portion out from 870m from shore. Along this inner section, dumping rubble on the seaward face continued at a rate of about 20,000t per year. Since this decision (presumably 1870 or 1871) and completion of the book (1883-4), the superstructure breached from 870 to 945m, from 975 to 1060m, and from 1286m to the end at 1426m.

Another example in Chapter 15 was Manora (Karachi), constructed 1869 - 1873. Again a low mound to 4.6m below LW, the superstructure was formed by 27t concrete blocks. It appears that wave breaking over the low mound was however somewhat similar to Alderney, and the superstructure was damaged by monsoon storms each year from 1871-1874. Open joints were filled during the repairs, and damage did not persist. At Madras, two breakwaters 1180m and 1210m long were started in 1876 and 1877 to a similar design by W. Parkes. The mound was taken up to 6.7m below LW surmounted by (again) 27t concrete blocks, jointed by mortice and tenon joints. The harbour was hit be a cyclone in 1881, just before completion, which failed almost the entire outer lengths of the superstructure running parallel or slightly oblique to the contours. The inner lengths perpendicular to the coast remained undamaged. At the damaged sections, the seaward foundation blocks had been undermined by erosion of the mound, whilst the harbour side blocks were (probably) damaged by overtopping pressures / flows. At this point Vernon-Harcourt has a bit of a rant about the width of the superstructure at Madras which he reckons to have been much too narrow in comparison with other breakwaters. He also argues with Parkes' assertion of some "*novel phenomenon of cyclonic disturbance raising waves of unprecedented power"*. Vernon-Harcourt asserts that it would be sufficient to design the superstructure wide enough, and bond it transversely to ensure a stable solution. Three alternative proposals for reconstruction were prepared: by Parkes; by Molesworth; and by a Joint Committee of Hawkshaw, Coode and Stokes, in increasing order of costs.

The Western breakwater at Colombo, designed by Sir John Coode to an intended length of 1222m, was started in 1873. It had a somewhat similar configuration as Alderney with a superstructure formed by two walls (sea wall and harbour) founded on a low rubble mound, and filled between by rubble. The mound was brought up to 6.1m below LW, and the concrete wall blocks (16 – 33t) were placed as slice-work by a steam Titan. The main length of mound was placed from a steam-powered hopper barge carrying an average load of 43t. The upper levelling course was dropped from hand barges carrying about 6.8t, dropping into hollows from the initial placement. Divers were used to level the foundation before block placement. Construction generally halted for the south-west monsoon, middle of May to October. The seawall (to 3.3m above LW) was advanced ahead of the harbour / quay wall until in the 1878 S-W monsoon 140m of the seawall slid backwards by about 0.350m under repeated wave impacts. Thereafter the harbour wall was advanced with the sea wall. It was planned to add a parapet wall of a further 3.3m height once all settlement of the mound and superstructure had been absorbed.

During the period in which the harbour design had been developed, trade at Colombo expanded by more than 2x, suggesting that the harbour might usefully be expanded in area. John Kyle, for the Colonial Government, proposed in 1877 to extend the western breakwater, and to add a northern breakwater (1710m). It was noted that waves were not large, generally limited to 0.6 – 1.5m, occasionally reaching 3m, "*but they break with great violence over the breakwater*" superstructure. So even with the 3.3m high parapet wall to a crest level of +6.6m LW (or 6.0m above HW), overtopping was too great for the intended use of the leeward face as a quay.[93] It is clear that engineers were starting to understand that wave severity was not necessarily linked to the extreme wind speed at the site, and that fetch length may be important, see methods to predict wave heights by Hawksley or Stevenson discussed in Chapter 1.

Vernon-Harcourt then discusses several changes to the breakwater layout and section that were possible / anticipated for Colombo, particularly those changes required to meet a budget ceiling of £800,000. The plan was to discontinue the separation between sea and quay walls, forming a single wall of 11m width, required solely to provide wave shelter, and not to also act as a quay. Various configurations of northern breakwater were considered, but all would take the total cost above the allowed budget, so they appear to have been abandoned. The outcome is not however recorded.

At the mouth of the River Tyne, James Walker designed in 1853 two piers or breakwaters (of 640 and 1285m) to entrain and shelter the navigation channel. The initial design resembled closely those used by Walker for Alderney and St Catherine's. The superstructure was founded at LW, and formed by two block walls, filled between by rubble. The seaward mound was raised to about mid-tide level. Construction started in 1856, and rapidly suffered the damage that would be anticipated from experience at Alderney. In 1863, it was clear that the mound at LW was not stable, so the foundation was lowered to 3.7m below LW. But in December 1867, storm action destroyed 73m of the sea wall on the North Pier, and a similar length of the inner (harbour) wall of the South Pier. Along the north side, the foundation mound, protected by 5–10t rock was lowered by wave

[93] One might deduce that long-period swell waves were breaking over the rubble mound onto the superstructure giving impulsive loadings, and overtopping. But nowhere in these discussion does the author discuss wave periods, or wave shoaling.

action to 5.2m below LW. The engineers on the project, Ure and Messent made a number of important changes to the design, particularly: lowering the foundation to 4.9 – 6.4m below LW; protecting the foreshores by 36t concrete blocks, and concreting the hearting between sea and harbour walls. In effect, the foundation mound to the main breakwater wall was reduced to little more than a metre high over much of its length. Up to 1883, the walls had been formed from staging erected on piles buried in the rubble mound. For future construction work (after publication of this book), Messent had designed and constructed a Goliath block-setting crane to operate from the superstructure, operating to set superstructure wall blocks, and to place the 36t armour blocks to protect the seaward mound.

**Chapter 16** describes harbour protected by vertical wall breakwater, including Dover, Whitehaven, Newhaven, Fraserburgh and Buckie. At Whitehaven, Smeaton proposed an enlargement of the harbour in 1768, but the work on a new west pier only really started in 1824, completed in 1839. The west pier was 310m long, founded approximately 2m below LW (tide range 7.9m). Each pier was formed by two masonry walls, again in-filled by rubble. The foundation to the west pier is not identified, but for the north pier it is identified as a "*concrete foundation placed on sand"*. Ramsgate harbour was the earliest use of solid vertical wall breakwaters formed by masonry. Vernon-Harcourt's main interest being the use of tidal flushing basins to control siltation, the use of a diving bell for the wall foundations, and the construction of artificial beaches within the harbour to reduce effects of swell.

In discussing Dover, the reader is reminded that the current harbour was mainly formed by Pearson to the design in 1895 by Coode, Son & Matthews, see Wilson (1919), so too late to be covered by Vernon-Harcourt's book. He does however review the alternate designs submitted around 1845 for a major defence harbour or of refuge, noting nearly 40 years later, that the Western Breakwater was only partially constructed with neither eastern or detached breakwaters having been commenced. Curiously, the vertical walls were to have been constructed of brickwork as experience of concrete blocks was very limited, but this is not mentioned further, whereas construction with 8t (?) concrete blocks is discussed.

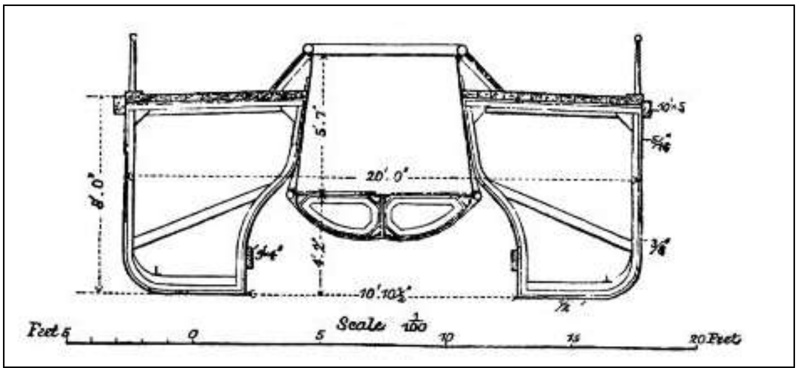


**Figure 11.7 WD Cay's hopper barge to lay concrete bags**
*First used on North Breakwater, Aberdeen*

Vernon-Harcourt notes use of diving bells at Dover (rather than helmet divers) to level the chalk foundation, and to lay the lower courses of the pier. He cites the advantage of visibility not being influenced by turbid water, isolation from (some) currents and wave effects, and the general absence of the cold felt by helmet divers, particularly in winter. He notes however that it does impose a limit on the blockwork size that can be handled. In discussing the potential seaward extension of the Western Breakwater, the author rehearses arguments on the need (or otherwise) for granite facing on the wall faces, with Mr Druce, the harbour engineer, not seeing the need for the granite. This argument re-appears in the discussion to Wilson's paper in 1919, and was firmly rebutted by Wilson. Vernon-Harcourt also notes that reducing the parapet wall from a thickness of 4.1m to only 1.7m led to its displacement under direct wave attack, although not where waves were oblique to the wall.[94]

Vernon-Harcourt notes the rapid progress at Aberdeen, occasioned in part by the use of concrete bagwork to form the foundations by WD Cay, see Figure 11.7. A similar approach was underway at the time of the book for Newhaven, using larger (104t) concrete bags, and then casting the mail wall between formwork. At

[94] See calculations in section 6.4.6.

Fraserburgh, the same construction method and equipment, including the concrete bag barges, were used as at Aberdeen, presumably for the Balaclava Breakwater, see Paxton & Shipway (2007). And similar construction was used at Buckie, although Vernon-Harcourt draws parallels with work by Poirel rather than WD Cay who had designed the extension, again see Paxton & Shipway (2007).

In his concluding remarks on simple vertical breakwaters, Vernon-Harcourt notes the laborious task of levelling the base, as at Dover, commenting that it was cheaper to form the foundation using concrete in bags at Aberdeen, Newhaven and Fraserburgh. He notes the gradual replacement of stone masonry by concrete, even if faced by granite. He also notes the advantages of using concrete (in formwork) to form the superstructure. In commenting on a low mound composite configuration that "*the only advantage an upright wall possesses over the mixed system … consists in the saving in material and freedom from settlement*", Vernon-Harcourt shows the ignorance of the time on the severity of impulsive responses, the potential range of wave conditions, especially of long wave periods, and the inability of his era to compute near-structure wave transformations.

**Chapter 17** discusses example harbour configurations on sandy coasts.

Not on breakwaters, but sometimes relevant to the materials and technologies available, in **Chapter 18** Vernon-Harcourt discusses lighthouses. He notes different approaches to selecting the initial batter at the base, arguing that hyperbolic or similar curves may encourage wave run-up, leading to heavy spray around (or over) the lantern. For Lighthouses formed on rocks or reefs, he notes that the native rock "*naturally consist of the hardest kinds of rock, as they … have longest resisted the action of the waves*". Preparation of the foundation therefore primarily involves removal of unsound rock, and in forming level benches to receive the lower courses of blocks. Blocks are cut to fit tightly, often dove-tailed both horizontally or vertically. Jointing can also use joggle bags, or sometimes steel bolts or iron ties. Some bases used concrete, but most used stone blocks cut to shape for both foundation and the main tower. In some instances, cofferdams were formed by sandbags, or blocks set in Medina or Roman cement. In **Chapter 20** primarily on dock walls, he discusses projects using lime mortars or concrete, and Portland cement concrete, but fails to give dates so it is not possible to decipher the progression of change in time.

**Specific (technology) examples:**

Thomas Stevenson's "Marine dynamometer" (p27-28).

William Dyce Cay's barge for placing concrete bags (pp 133-134)

Failure at Wick 1872, (p31-32).

## Cherbourg

Lack of a natural harbour on the Channel coast of France directed French military leaders' attention to (protecting) the bay at Cherbourg. A system of three breakwaters were first mooted as early as 1665, but only commenced in 1783 by the start of construction of the central breakwater. The breakwater design by De Cessart used a series of timber cones, set abutting each other at the toe. Each cone was intended to be 46m in diameter at the seabed, 20m diameter at the top, and 20m high. The cones were then to be filled by stone over the lower part, and masonry-faced concrete on the upper part. The concept did not survive early construction storms which severely damaged or destroyed many of the 18 cones placed. These relict cones were eventually cut-down to low water in 1789 and incorporated into a rubble mound to about 1:3 on the seaward face, 1:1 on the lee side. As at other sites, wave action eroded the seaward slope down to about 1:10 over the upper part to about 4-5m below LW. In doing so, the crest level became reduced, rendering it ineffective for carrying gun batteries protecting each end. Large stones were used to raise and protect the crest in 1802–1803 to accommodate these gun batteries. This rock was however damaged by storms, and it was decided in 1811 to take the foundations of the battery down to LW, and to construct as masonry protected by granite facing. Some

13300m$^3$ of "*the largest stone procurable*" [95] was placed in 1811, but by 1813 the works were stopped before completion. That work recommenced some 11 years later. Works to raise the main breakwater crest above water didn't start again however until 1830, again using the design around the central battery.

A row of concrete blocks cast in place on the rubble formed a toe / foundation. The lower slope was protected by large stones down to -5mLW at a slope of 1:5. The new superstructure caused uneven settlement in the somewhat variable mound, so the final part of the superstructure was delayed "*3-4 years to allow the mound to consolidate*". The superstructure of the central breakwater was complete in 1846 and the pierhead forts in 1853.

## Portland

Vernon-Harcourt discusses Portland Harbour in Chapters 10 and 12. Again a 'roadstead' harbour formed in a natural bay in the shelter of the Isle of Portland, the harbour was created initially by two breakwaters: the inner breakwater connected to the island; and an outer detached breakwater to the north-east with a 120m wide entrance. Both breakwaters were constructed as rubble mounds with superstructures from LW. Construction used Portland stone, run-out onto the breakwaters by timber staging from 1849, extending over the intended gap between the inner and outer breakwaters.

Vernon-Harcourt gives details of the staging and railway used to carry stone down from the quarry on Portland and out onto the breakwaters. The timber staging (piles about 10m apart surmounted by creosoted cross-beams about 5.5m above HW) was founded on iron screws into the clay bed. The stone was dumped in 'ridges from the staging, "*the waves gradually levelled these ridges*". Large stones (3-7t) were included in the material dumped at an average of 500,000t per year from 1853 to 1860, reducing to 140,000t per year to 1866, giving a total of 5,800,000t. The outer breakwater was then completed by two pierheads formed in masonry founded at -7.3mLW.[96]

## Dover

Vernon-Harcourt discusses the proposals to form Harbours of Refuge and notes that Dover was selected. Eminent engineers reported to the Admiralty on the location and sections of breakwaters to form a harbour of some 520 acres (21km$^2$) out to 7 fathoms (12.8m). The outer breakwater was to be aligned with the tidal currents (to reduce siltation). Eight designs were submitted:

- Sir John Rennie suggested a single detached breakwater, cranked landwards at each end, as at Plymouth. He also proposed a rubble mound section, again as at Plymouth.
- Mr G Rennie conformed to the anticipated three-breakwater layout, and also used a rubble section as at Plymouth.
- Mr Cubitt suggested similarly a rubble-mound three-breakwater solution.
- Mr Vignoles selected a rubble mound base, surmounted by a vertical superstructure (wall), again for a three-breakwater layout.
- Colonel Jones differed in suggesting two breakwaters with a single south-facing entrance with a composite breakwater, wall on low rubble mound.
- Mr Walker, Mr Rendel and Captain Denison de facto agreed in proposing the three-breakwater layout formed by upright breakwaters.

The Commissioners reported in 1846 in favour of Mr Rendel's design. Vernon-Harcourt noted damage to sloping solutions at Cherbourg and Plymouth, and the lack of suitable stone at Dover. He also notes the shortage of experience in concrete which "*existed at that time*". But given the chalk bottom, absence of local rock, "*and a moderate depth, the upright wall was the best system to adopt*". Vernon-Harcourt then notes that 40 years after the design had been approved, the western breakwater was only partially constructed (at 640m

[95] Vernon-Harcourt does not identify rock armour sizes.
[96] The two northern breakwaters were added to reduce the risk of torpedo attacks, but after this book was written.

length) and layout details were still “*open for further consideration*”. One aspect of significant progress in the extension of the Western breakwater was in the manufacture and placement of concrete blocks which had already shown the promise of “*greater economy in construction*” ...”*as the reliable character of well-made concrete blocks is so fully established*”.

## 11.13 Shield (1895)

A handbook of 17 chapters covering harbour design, based significantly on the author's experience as Executive Engineer at Peterhead Harbour of Refuge. Chapters 1-6 cover wind, waves, tides currents, wave exposure and quarrying, then materials (stone, concrete, cement, timber, iron and steel in Chapter 7. Different purposes / formats of harbours are discussed in Chapters 8-10. Breakwater types are discussed in Chapter 11, pier-heads in Chapter 12, and wave screens and floating breakwaters in Chapter 13. Chapter 14 describes breakwater construction, Chapter 15 discuses foundations and 16 issues related to settlement, particularly as affecting masonry. Chapter 17 cover siltation.

In reviewing this book, it is impossible to avoid making comparisons with Vernon-Harcourt's similar book of 10 years earlier, indeed those contrasts are of themselves seminal in illustrating the areas and speed of change of knowledge and understanding, or perhaps highlighting Vernon-Harcourt’s short-comings in analysis. It is clear that Shield understood many wave processes rather better than Vernon-Harcourt did, and that he used his personal experiences in South Africa and at Peterhead to very good effect. The book itself is however under-edited, with excessive duplication and/or material misplaced. Further significant weakness are the frequent confusion or inversion of the terms 'force' and 'pressure', use of 'strain' when meaning 'load', and the failure to clarify the origins of some calculations.

**Chapter 1** starts with wind, "*the generator of ordinary sea-waves*". Shield notes that the "*working season … during which gales are least frequent and the sea smoothest*" varies in different parts of the world and by season. He shows detailed example records of wind measurements from Peterhead illustrating direction, season, and severity (Beaufort scale, originally devised in 1805), firstly using an occurrence wind rose, then with a modified diagram of his own including the wind direction, frequency, and speed (Beaufort). He illustrates a discussion on wind forces with measurements from the Forth Bridge, and illustrates the frequency of gales around the British Isles with statistics from the Met Office. He describes example meterological occurrences that presage storms, and notes the potential for swell to precede a storm noting and example in Bermuda in 1839 when swell arrived "*a full three days before the storm*".

On waves in **Chapter 2**, we see evidence of work by Airy, Scott Russell, Weber in the descriptions of wave motion, with significant emphasis on wave transformations and breaking. Noting the transition from deep-water waves to "*waves of translation*", Shield identifies that breakwaters at Holyhead, Portland and Plymouth may cause waves to change "*for the worse*"…"*the waves therefore break upon the mound*", although for those breakwaters he believes that they do not then "*reach the superstructure … in their solid form*".

For Alderney, he does however identify that the combination of water depth over the mound, and the incident wave conditions, leads them to "*break just as they reached their greatest destructive power*". He also notes that there are "*several features in the design of the Alderney breakwater which … experience has shown to be objectionable, and which doubtless contributed to its destruction*". In further support of this, Shield analyses a failure at Holyhead where differing levels of the mound in front of the wall led to dramatically greater wave loads.

He discusses conditions of spilling breakers, termed 'cresting'. He also notes processes of shoaling and breaking as waves pass over reduced water depth, noting their ability to revert to form once the shoal / reef has been passed. He cites a wave velocity formula which may be written:

$V = \sqrt{(gd)}$ where $d$ is the local water depth

He then diverts himself (briefly) to discuss waves caused by earthquakes (tsunamis), from which he returns to discuss swell, and thence to the maximum height of waves. He quotes Lord Dunraven as stating that waves (one assumes $H_{max}$) off SW Ireland have reached 46m. Captain Belcher (HMS Bellerophon) records having met waves of 27.5m in the North Atlantic, corroborated by observations near Peterhead (nearshore) of waves of 27m to 30m, yet he then downgrades his estimate of "*the greatest height of storm waves, trough to crest, does not exceed 50ft*" (~15m), except around Cape Horn where he acknowledges that 60ft (~18m) may be possible. For deep water, he cites a formula by Thomas Stevenson for the height of waves in feet ($H_{(ft)}$) in heavy gales:

$$H_{(ft)} = 1.5 \sqrt{F} \qquad \text{where } F \text{ is the fetch (in nautical miles)}$$

Or modified for short fetches:

$$H_{(ft)} = 1.5 \sqrt{F} + (2.5 - \sqrt[4]{F})$$

Shield notes the existence of another formula by Mr Hawksley, but that its results differ significantly from those from Stevenson's. He quotes various (Indian ocean) wave observations from his time at Port Elizabeth, South Africa. At Peterhead, Shield recalls waves of $H_{max}$ ~8m, $L$ ~ 150m, in 13-15m, then starting to break at the -10m contour (not unreasonable for $H_b/h \approx 0.78$). He later imagines this wave hitting a wall, rising to just under 8m, and estimates a '*hydrostatic pressure'* of 1622 lb/ft$^2$ (78 kPa). He then contrasts this with a 'dynamical force' of 6083 lb/ft$^2$ equivalent to 292 kPa.

On erosive coasts, he cites observation by Coode indicating that waves shorter than $T$<7s may be erosive, whilst those longer than $T$>8.5s may be accumulative.

In considering beach material, he notes the tendency for wave action on a shingle beach to "*be absorbed by filtration*", and that the "*power of drawback being greatly diminished*".[97]

**Chapter 3** describes Tides, not of direct concern here, but Shield does discuss the differences of geometry / volume required for vertical or rubble mound breakwaters under different tidal ranges. **Chapter 4** describes currents, touching on their effects on navigation and on siltation.

**Chapter 5** then re-visits fetch, exposure and wave-power. Shield notes the ability of waves to diffract around headlands, to refract towards contours and over sunken rocks and similar bed features, for fetches to be limited by the size of the storm, and *de facto* that strong winds may not blow from particular directions. He also notes instances where wave penetration is worst when strong wind veer, carrying previously large waves into a harbour sheltered from their original direction.

He perceptively notes that early stages of a storm may cause significant local deepening (scour), noting 1.5-1.8m scour at Port Elizabeth, 3m scour at IJmuiden.

Whilst using the term 'wave-power', he then discusses manifestation of wave forces, noting the movement of 90 tonne boulders in Peterhead Bay, but then ascribes these to glacial action! Shield then notes the effects of heavy overtopping and down-fall pressures, noting effects of waves on ~ 37m high cliffs of Caithness that had eroded a depression 3.7m deep and 33m wide, and ascribes damage at Alderney and Wick to similar causes. He notes instances of overtopping water reaching 30+m high, and ascribes a down-fall velocity of ~ 25m/s, suggesting that the force (pressure?) on the breakwater roadway might be 4 x that on the front face. [98]

Considering the breakwaters at IJmuiden (his spelling Ymuiden), he notes various 20 tonne concrete blocks displaced onto the top of the pier, or over it. He also cites full courses of blocks (jointed together by wrought-iron cramps) displaced into the harbour. At Wick, he accepts stones of 8 or 10 tonnes thrown over the parapet

[97] Whilst this chapter was still in need of technical checking and some editorial rearrangement, the content suggests that Shield understood many wave processes significantly better than had been the case 10 or so years earlier demonstrated in Vernon-Harcourt's book.
[98] The origins of these calculations are not given.

(6.4m above HW), but regards the displacement of 1350 or 2600 tonne concrete blocks as (at least in part) driven by foundation failure, and quotes Mr Parkes from ICE proceedings (vol. 63, p52) as regarding it "*as so extraordinary a result*".

**Chapter 6** considers quarrying, insofar as it may be useful to the harbour engineer. Shield defines a number of geological terms, and gives some guidance on blasting, particularly the amount of 'powder' required for different types of rock, often around 4-6 tonnes of rock for each pound of powder, so about 10 tonnes of rock per kg of powder. He discusses the use of single very large blasts and contrasts with small-blast firing. He also discusses the types of equipment available to quarrying operations, primarily cranes and drills. Two main types of drill are described. 'Jumpers' are lifted and struck against the rock, they generally used sharpened ends, and were often used 'wet' where the cutting dust / fragments acted as a cutting paste. Shield also describes steam- (and hand-) driven Ingersoll drills with cross-heads, contrasting the hole diameters (larger for steam drills), set-up times (quicker for hand-drills), the needs for drill sharpening or re-steeling, and rock production rates, and concluding that "*the cost of drilling by steam and hand, per ton of stone, was therefore about equal*". He notes that drills in hard rock usually require re-sharpening for each 0.6 to 0.9m bored.

On blasting, he notes that dynamite, guncotton and other 'quick' explosives can leave cracks far from the point of explosion – blast fractionation. For the production of rubble this may be an advantage, but can be a major problem for block-stone for ashlar, and their "*use is inadmissible*". For granite ashlar, Shield recommends 'seam firing' in which an initial explosion moves a rock mass a small distance, then larger powder charges are tamped into the resulting seam to "*heave out*" the rock mass. He also discusses the need and materials for 'tamping' re-sealing the drill hole above the charge.

**Chapter 7** is titled 'materials', but does divert itself into causes of erosion, including movement of shingle. The main disrupting agents are erosion by sand or shingle, rain and frost, acids in the atmosphere and in sea-water, excessive heat, and ice. He particularly notes the erosive action of sand and shingle, requiring use of highly wear-resistant materials. He also notes effects of 'wave-stroke' [99] on joints and seams, noting that this may move large stone or concrete blocks. He also notes the process of onshore stone movement caused by 'kelp rafting'.

The second section of Chapter 7 describes the types of stone starting with lists of example densities. He discusses use of granite, syenite and hornblend noting their similarities and differences. He also identifies their chief mechanisms for deterioration, primarily by decomposition of the feldspar crystal, by water absorption into mica, and oxidation of iron elements. He notes the susceptibility of limestones and sandstones to boring molluscs, primarily *Pholas dactylus*, or *Saxicava*. He notes the hardness of metamorphic sandstone or quartzite, and its potential advantage in marine structures, but notes that "*working it is almost impossible*". This section is concluded by a short discussion on the use of blast furnace slags as a substitute for rubble and/or armour.

The third section of Chapter 7 covers concrete, curiously in advance of the later section on cement! He notes that use of concrete has only been prominent in "*the last fifty or sixty years*", and notes that: "*English engineers seem to have little faith in lime for sea-works, seeing that almost without exception, they now employ Portland cement*". In contrast he notes use by French and Italian engineers of lime rendered hydraulic by addition of aluminium silicate. He observes that use of lime mortar at Holyhead was a failure, even (or especially) with the addition of pozzolanic material, with significant loss of mortar from joints requiring raking out and replacement by Portland cement mortar. He notes however successful use of lime mortars in dock works at Liverpool, London and Glasgow (all ports where docks are sheltered from waves and significant current effects).

[99] Perhaps now termed 'wave impact'.

Concretes are mixed by hand, or by a number of different machines, for which he cites: Coode's; Messent's; Ridley's; Le Mesurier's; and Carey-Latham's, describing the main principles of their action and highlighting notable features. He observes that a very large version of the latter machine achieved 300 tonnes per hour at Newhaven (perhaps 125m$^3$ per hour), but that the more common "20-yard mixer" machines usually delivered around 11-12m$^3$ per hour of mixed concrete.

For concrete blocks, timber formwork may be withdrawn after 3-4 days, although blocks should not be lifted until 3-4 weeks old. At IJmuiden, blocks were not incorporated into the breakwater until around 3 months old, face blocks being formed of:

| | | |
|---|---|---|
| 1 part Portland cement | 3 parts sand | 5 parts gravel. |

At Aberdeen, blocks for the south breakwater were formed by:

| | | |
|---|---|---|
| 1 part Portland cement | 4 parts sand | 5 parts gravel, occasional pieces of rubble |

And at Fraserburgh, proportions varied, OPC : sand and gravel:

| Initial, bagwork | Bagwork, currents | Mass, summer |
|---|---|---|
| 1:7 | 1:4.5 | 1:9 parts |

In a paper to ICE (Vol 87, p131) Willet expressed concern over the strength of such concrete: "*I am convinced from recent experience that the proportion of 1 of cement to 9 of gravel & sand adopted at Fraserburgh is too weak for such exposed works. The proportion adopted for Sandhaven and Portsoy of 1 of cement to 6 of gravel and sand has been proved to form concrete of more suitable strength.*"

Shield then reviews working with concretes using Roman or Medina cements.

The fourth section of Chapter 7 discusses cements, chiefly Portland cement, quoting at length from an ICE paper by Bamber (Proc. ICE, Vol 107) on the manufacture of cement from clay and chalk. He discusses at length various tests to assure himself of the properties of the cement, this appears to significantly predate any British Standard on OPC. Shield requires cement for marine works to fulfil:

1) Be uniform dark grey;
2) Specific gravity ≥ 3.14 when fresh, ≥ 3.125 a month after being ground;
3) Finely ground, 92% passing 2500 meshes / inch$^2$;
4) Time of setting 0.5-1.5 hour at 60°F (16°C), fresh cement;
5) Slabs 1/4" (6.35mm) thick, stored in water for 24 hours, when heated to 160°F (71°C) should not show signs of 'flying';
6) Briquettes of 1.5" x 1.5" (37.5mm square) should stand a tensile load of 450-600 lbs (2.0 - 2.7kN) at 2 days, 700-900 lbs (3.1 – 4.0kN) at 4 days, and 950-1100 lbs (4.2 - 4.9kN) at 7 days.

He then describes Roman cement as produced from Sheppey, Yorkshire and Harwich, noting it to set very quickly, but being weaker (especially if mixed with sand) and more expensive than Portland cement.

The fifth section of Chapter 7 describes timber as used for marine works, noting particularly their propensity to damage by sea-worms or borers. He identifies commonly used timbers:

| | | |
|---|---|---|
| Baltic redwood | Pitch pine | Oregon pine |
| English oak | American oak | Teak |
| Greenheart | Jarrah | American rock elm |
| Black ironwood | Sneezewood | |

He identifies key sea-worms as: *Teredo navalis*; *Pholas dactylus*, *Limnoria terebrans*, and *Chelura terebrans*, and discusses the utility (or not) of various treatments to deter worm attack. He emphasises that creosoting must include all drilled or cut surfaces, particularly for redwoods, pines and oak. Greenheart and Jarrah are more resistant to borer attack, but should be reinforced by hoops to prevent splitting during driving.

The sixth section of Chapter 7 describes use of iron and steel, noting that they are not common in breakwaters, but have been used in landing-pier and related works. Noting that other literature cover production and properties of iron and steel, he focusses on durability and practical uses in marine structures, although noting that it was then "*impossible to lay down any rule for the rate of oxidation of metals in seawater*". He notes that ICE papers in 1884 and 1885 by Andrews described galvanic corrosion of various metals over about 4.5 years. Shield however then contrasts corrosion to the wreck of the SS Gambia in Algoa Bay, South Africa showing corrosion rates to the wrought iron of about 7x greater than Andrews' laboratory experiments.

**Chapter 8** turns to "considerations affecting the general design of harbours", that is mainly on their siting and plan layout. Shields highlights the need for good survey data, sources of constructional materials (and fresh water), space for construction and storage yards, availability of labour. He notes the need to identify the number, size, draught, type and purpose of vessels to be accommodated, trades to be served, availability of roads and railways, beaching grounds and spending beaches. Given his work at Peterhead, he particularly reviews requirements for 'harbours of refuge', noting that the 'refuge' may be from an enemy as well as from storms.

**Chapter 9** is mainly on "estuary harbours" although it includes many examples from open coasts. It is primarily concerned with plan layout, especially with reference to siltation, harbour entrance widths and diffraction of waves into harbours.

**Chapter 10** then describes "fishery harbours", including the use of small basins stop-logged against wave penetration, and the need for fish landings to fit with train departure times. He illustrates the need for all-weather, all-tide harbours by calculations of income improvement.

**Chapter 11** describes various types of breakwaters (p172-224), distinguishing here between 'vertical' and 'mound' types. He observes that vertical types take less material, but implies that large water depth or soft foundation materials might favour mounds. He particularly notes that 'translatory' waves may be especially destructive to vertical walls – "*vertical-faced work cannot be regarded as a charmer of such waves … a vertical face therefore fails to uphold all the good qualities which are so often claimed for it, just at the time when a display of them would be most acceptable*".

Timber cribwork breakwaters have been used at various places, often for works of temporary nature, to be improved or extended later, given their propensity for worm or borer attack.

Vertical walls of concrete or masonry blocks and dry rubble hearting have been used at many places where depths are small and tides allow reasonably easy access to found blocks onto the seabed. He cites Kilrush, illustrated, and Wick where he opines that the failure started from down-fall pressures failing the lee-side wall first, the seaside wall lasting some time even after loss of the hearting. He notes also the failure of St Helier landing pier by the same cause.[100] Shield concludes that "*there can be little doubt that loose rubble hearting is to be avoided*".

Shield asserts that breakwaters formed by large concrete blocks in courses possess many advantages, particularly as block-making can proceed even when breakwater construction is inhibited by storms. Blocks in courses do however require precise and robust founding layers. The main difficulty had been the need to lay the underwater foundation layers 'dry', that is un-mortared, so without the adjustment of level possible with a mortar. These difficulties are exacerbated by any differential settlement, perhaps requiring blocks to be dressed in-situ. Delays also occur when weather and unfavourable tides interrupt the setting in mortar of the lowest cemented courses. On local settlements, Shield notes that the advance of 'Titan' block setting cranes

[100] This failure mode might be similar to the failure in 1879 of the breakwater at Greve de Lecq on Jersey, see 2.12.

over recent work may lead to uneven settlement, "*is often responsible for a good deal of trouble in this respect*". A footnote records that the Titan crane at Colombo weighed 180 tonnes, and caused ~ 0.1m settlement "*in passing over new work*". On a related issue, he notes a local failure at Aberdeen where dry set blocks 'fell away' from monolithic work above, being relieved of the weight from courses above, and therefore easy for wave action to draw out the loose blocks. He notes here that the sloping-block system, otherwise termed slice-work removes many of these problems, thus allowing construction to proceed irrespective of tides.

On concrete bag-work in foundations, he advises laying bags athwart the breakwater, and filing interstices with small bags of "*especially soft rich concrete, or with similar concrete deposited in mass*". Blocks should be laid as 'headers' to give the smallest possible area exposed to wave pressures. Batter of the face "*is objectionable as it… relieves the outer blocks of the lower course from weight*", and requires production of many different sizes. His favoured form of wall uses concrete in mass up to neap low water, with concrete blocks set in cement (mortar) above. This avoids difficulties in forming "*mass-work within the tidal range*".

Shield then discusses breakwaters and piers formed by mass concrete using formwork, "*depositing freshly mixed concrete within temporary timber frames, either under or above the water level*". He notes that this may generally require relatively little expenditure on plant, or alternatively such expenditure may deliver rapid progress and reduced (relative) cost for larger projects. This approach delivers (immovable) large monolithic blocks. The disadvantages include this being a 'fine weather' method, subject to damage by storms to formwork and fresh concrete, and subject to interruptions of progress.

Methods to deposit concrete underwater included skips with hinged flap bottoms, by tremie pipe (then termed a 'shoot or trunk', or by 'tripping' bags with a (closed) opening in the base from which concrete can be deposited as close to the bed as possible. He later elaborates on the use of 'bag-blocks' using iron frames with opening bottom doors, and notes Dyce Cay's use of an adapted hopper barge at Aberdeen. He discourages dropping bags from any significant height, and advises against the concrete being "*meddled with*" during setting. He illustrates the use of a bag-work foundation and an upper structure of mass concrete in frames by a section of Fraserburgh breakwater.

Noting instances of failure due to poor concrete, he again concludes that 1 part Portland cement to 8 or 9 parts sand / gravel is likely to be too weak, preferring 1 part Portland cement to 4 or 5 parts sand / gravel / extra small broken stone. He suggests that monolithic work should be in sections of 4-6m length, each terminated by a vertical joint. As an aside, Shield notes use of 300t concrete blocks at Dublin by Stoney, placed by floating shears, but concludes that this method is not suitable for exposed sites.

Citing construction at Manora (Karachi), Mormugao, Reunion, Colombo, he then discusses use of slice-work. He notes use of blocks generally of 16-40 tonnes (113 tonnes at Reunion), inclinations of 68 to 76°. He notes some propensity for the inclined face to slacken, 'outrunning' the crest, noting the need to ensure that the inclined 'columns' remain in intimate contact with the previous such column.

He notes that the use of the concept was not then new, recalling use of inclined stones at Ardossan, early 1800s, the first phase of Aberdeen North pier by Telford & Gibb in 1812, and Peterhead east pier by Telford in 1820, see discussion in 3.7.

Turning to mound breakwaters, he reminds the reader of the influences of rock availability, and of tidal range, in determining the mound section. He notes the propensity for dumped rock to sit at 1:1 to 1:1.25 below 3-5m below MLWS, yet for the upper slope to lie 1:12 (c.f. Holyhead) or perhaps 1:5 (Plymouth). He notes that the stone on some of these shallow upper slopes are then pitched to put "*a stop to the disturbance of the rubble*", but that the resulting smooth slope increases the wave draw-back, thus causing more disturbance at the toe of the pitching. He observes that the stones on the seaward face are subject to wear under wave action, but judges "*that the annual waste from this cause, even … at Holyhead, is very small*", suggesting that "*works of this class may be maintained at a moderate cost*", perhaps a rather over-optimistic view for some such structures!

Shield then draws the rather important conclusion that a mound composed of rubble, say 0.5t upwards, "*will remain undisturbed in heavy seas if no superstructure be placed upon it*". He then moves directly on to discuss Alderney, noting that the rubble mound had been left for 3 years to 'consolidate', and remained undisturbed in heavy winter storms at 5m below MLWS. Yet the addition of the breakwater superstructure caused substantial downward action by reflected wave action, "*so great that it ploughed out the mound to a depth of 20ft below low water*" (perhaps 6m below MLWS), for a distance 80-90ft from the wall (~24-27m). Qualitatively similar movement of stone at the wall toe occurred at Madras (now Chennai).

The solution to this conundrum at Holyhead was to take the rubble well above HW, and maintain it, or to keep it well below the disturbing action of waves. He concludes that "*there can be little doubt that failure to recognise this* (at Alderney) *largely contributed to the destruction of that work*".

He notes that opinions differ on the desirability of using only larger rock in the mound, or using 'large and small sizes indiscriminately'. He accepts that the smaller fraction will be more mobile, and therefore contribute to the wearing away, but he opines that some smaller material is advantageous in filling interstices and improving mound stability, noting that at Holyhead voids occupied about 33%.

Turning to Alderney, Shield notes the absence of examples and guidance available 50-60 years previously, but he summarises the chief lessons to be learnt:

a) The wave action, quoting Sir John Coode –" *the exposure was great, but the very heavy sea against the pier was mainly due to the peculiar configuration of the bank and to the profile of the work itself*". Sir John Hawkshaw with Sir Andrew Clarke stated that: "*From the shape of the mound and of the wall, the seas were thrown up to a considerable altitude; in falling back again their momentum drew away the deposit from the base of the wall, and the bottom courses dropped down, having nothing to stand upon.*" Continuing wave action then washed out the hearting between the walls, leading to breaches through the walls.
b) Settlement of the mound, by 1/20 of its height amounted to 2m at the seaward end where the depth at MLWS was 41m. This settlement led to cracks / fissures in the superstructure within which air compressed by wave impacts forced out the face stones.
c) Loose or weak hearting.
d) The role of batter in unloading the weight of the parapet through the face stones, allowing their withdrawal.
e) The "*evil effect of parapets in increasing the recoil of the waves*", so the absence of any reduction of reflections by wave transmission.

Shield then reminds us that, in their report to the Board of Trade, Sir John Hawkshaw and Sir Andrew Clarke recommended, as a remedial measure, that the parapet or promenade wall should be removed in its entirety, and a wave breaker mound should be formed of large blocks along the seaward face of the breakwater, extending to slightly above HW. An alternative would be to feed enough rubble to raise the mound to above HW, the Holyhead solution.[101]

Shield then turns to breakwaters formed (mostly) by concrete blocks[102]. He cites examples at Alexandria, Algiers, Port Erin, Port Said, St Jean de Luz. He often refers to the blocks being "*thrown into the sea*" or placed "*pell-mell*" implying no systematic control of individual placement. He notes that a concrete block mound may cost 5-6 times that of the same volume of stone rubble.

**Chapter 12** is concerned with pierheads and (plan-shape) returns, illustrating four main types. He notes "*additional strength being given either by adopting a better class of masonry, by dovetailing or dowelling the blocks, or in some other way*".

[101] Jensen et al (2017) discusses use of a rock armoured slope over the outer face as a method to rehabilitate Alderney Breakwater in the 1990s.
[102] A solution adopted for some large Spanish breakwaters about a century later.

**Chapter 13** describes wave screens and floating breakwaters, noting with ill-disguised contempt the frequent hope that "*waves could be stopped or broken up and destroyed without the structure ... having to sustain the shock of the wave-impact; in other words that action and reaction are not equal*" .

Shield then proceeds to demolish the claims for the wrought iron louvred wave screen proposed by WB Hays, using terms as "*old-fashioned and long-ago-exploded notion…*". He similarly explodes the idea of M. Scott of perforated plates carried on counter-fort walls, themselves founded on a rubble mound, or the piled wrought-iron screen of EK Calver.

On floating breakwaters, he argues that they would need a beam able to cover at least 3 wave crests, so rather more than 2 full wave lengths, which he argues to be "*absurd*". He concludes that the design of floating breakwaters that had come to his notice were "*of so unpractical a character that it would serve no useful purpose to describe them*".

Happily, in **Chapter 14** on methods of constructing breakwaters, the author can shift into more optimistic, nay positive, mood. He discusses use of: staging to place materials; barges for rubble or concrete bag-work; over-end setting machines for blockwork, and cranes.

A major advantage of staging is to remove weather dependency from many operations, allowing dumping of rubble to proceed uninterrupted, to the great benefit of quarry productivity. Staging also allows easy and confident measurement of level and line for rubble mounds, improving the ability to deposit rubble of different classes (perhaps here a movement towards the modern layered rubble mound breakwater?). He describes various types of rail wagons used to discharge rubble, including GM Dobson's design of bottom-discharge rail wagons for Holyhead. He recommends that staging be elevated at least 6-7m above highest tide in exposed locations, and notes the success of Dyce Cay's staging at Aberdeen being at 10m above HWOST, and using round piles (to reduce wave forces?).

He acknowledges that staging is expensive, costing the equivalent of £65 per metre of breakwater at Holyhead (1895 prices), and thus adding £0.02-0.03 per tonne to the cost of dumping rubble, but he concludes that "*the advantages of staging are however so great that they will in many instances be found to outweigh its shortcomings*".

He notes use of hopper barges for dumping rock, but also barges carrying wagon bodies (rock skips) swung from cranes to place larger rock. He notes that blocks up to 200-300tonnes were placed on South Gare breakwater, River Tees, from floating shears carried on barges (Proc ICE, Vol 90, p352). [103]

The last section of Chapter 14 is devoted to cranes or (block-)setting machines. At this time, they all ran on rails, set into the staging or on the breakwater itself, and were powered by steam engines. Some machines were fixed in orientation, many include travellers or 'jennies', others slew / rotate (often termed 'radial' machines). Whilst never discussed *per se*, the impression is given that each such machine is a 'special', perhaps being similar but not identical to a version used elsewhere. There is no discussion of manufacturers or suppliers, merely of generic types. Indeed, the example used on the River Tyne piers is ascribed as being designed by PJ Messent, thus implying that they were probably built job-by job.

A range of method to lift blocks are discussed and/or illustrated, including T-heads, Lewis or screw bolts, expanding jacks, and scissor grips or clamps. A short section is given on diving, primarily comparisons for Dover and Alderney between use of helmet divers or diving bells, although his comments are not conclusive.

**Chapter 15** considers foundations, in rock, in sand, gravel or clay, or onto rubble mounds. In doing so Shield emphasises the importance of foundations for vertical walls being "*as perfect as possible*". Rock foundations

[103] An interesting omission is any mention of the motive power of any of the floating plant. None of the illustrations show propellers, nor emphasise steam winches, yet no mention is made of methods to warp barges into position, nor of steam powered tugs. Perhaps the latter were so commonplace by this time as to not require mention.

may initially be regarded as ideal, but can be "*beset with difficulties*". The formation of the joint between the (upper) blockwork and the (lower) parent rock is always difficult to form. Where masons can access the rock-head, the surface can be prepared to receive the foundation stones, although the choice of cementing for gaps and joints will depend on the time and frequency of inundation, and on the exposure to waves or currents. Blocks may be set around the periphery of the area to be filled by concrete. Alternatively timber formwork may be used to contain foundation mass concrete, planks and jute canvas being used to seal irregularities, or concrete-filled bags may be used around the periphery.

Noting difficulties in securing the "low-water course", Shield advises either finishing mass concrete foundations "*several feet below LW*" continuing upwards in blockwork, or continue the mass concrete work up to about 1/3 tide.

He then discusses construction on slopes, uneven foundations, gullies and pockets. He also diverts himself into re-running some of the arguments for and against over-end construction versus building the foundation layers from staging.

To save concrete, some engineers have proposed to form a 'ring-beam' of concrete bags, then level between with small rubble, bedding the blockwork onto the bags and rubble. Shield notes that this requires that both be accurately levelled and can quickly fail if the bags settle. At Hermitage breakwater, St Helier, Mr Kinipple reported: "*from the foundation area, many of the smaller bags, the whole of the broken stone, and some of the rubble and concrete filling between have been washed out… leaving cavities large enough to admit a diver …*"

Sand "*forms an excellent foundation, so long as it is not allowed to escape*". Any foundation for a vertical wall on sand must therefore be protected against scour. Such toe protection may need to extend significantly on seaward and leeward sides to resist effects of waves and currents. Muddy bottoms often require the mud to be removed, or to drive timber piles well into it to form support for a stone foundation. Driving the piles well below mud level will help protect against seaworms or borer attack.

**Chapter 16** discusses settlement and bond of blockwork, noting from the start that "*more or less settlement can seldom be avoided*". He notes that settlement can be ascribed both to yielding of the ground, and to consolidation / shrinkage of any rubble mound. Construction practice at the time was to leave the foundation mound for a sufficient time in the hope that natural exposure to wave and current loads would consolidate the loose placed rubble. It might also gain strength by accumulating sand in the interstices. Shield notes however that, however long the mound is left, the application of the weight of the superstructure is sure to cause additional subsidence. This is worse for mounds below (say) 6m below LW as even the heaviest seas will have given little consolidation, most being generated by the weight of the superstructure.

Uniform settlement is not harmful *per se*, but differential settlement is almost always harmful. At Holyhead the mound settled by about 1/15 of the mound height, but did so uniformly so the blockwork above was little affected. At Alderney, settlement was generally limited to 1/20 of the mound height, but the superstructure "*was particularly liable to produce unequal settlement*", and cracked. Shield notes that the construction of the Alderney superstructure proceeded in fortnights so that the masons would have LW access to cement the lower courses, and that these could then be secured by the weight of blocks above. The walls therefore had breaks at the division between each fortnight's work, "*between which unequal settlement of the base necessarily occurred; and a still more marked division was found at the close to a season's work*".

Shield notes that settlement can be accommodated, indeed subsequent courses can be adjusted, when using sloping blocks (slicework, see section 3.7). Bonded blocks may initially inhibit settlement, but beyond that differential settlement will induce cracking. It may be better to insert a joint to allow sections to settle differently, filling the joint later with grout. Bonded blocks may be jointed using dowelling or joggles, sometimes of stone well grouted, sometimes of joggle bags filled *in situ* by liquid mortar. He cautions against loose joggles as then can wear away their hole. Complex tabling and/or dovetailing can be used, but only if circumstances allow very close block placement. Blocks can also be secured by vertical dowels using short length or rail penetrating halfway into the course below / above. On the Tyne, PJ Messent used 'piano blocks', L-shaped in section, to prevent sliding of one layer relative to the other.

To finish, **Chapter 17** discusses (harbour) silting, describing the main processes in clear and simple terms. He distinguishes between suspended sediment and bed-load, and discusses the roles of tidally-driven currents and wave driven drift. Given the widespread nature of these processes, he notes that "*total immunity from dredging can therefore scarcely be expected*", but that a dredging cost of £22,000 to remove about 500,000m$^3$ per annum from the entrance to Ijmuiden could be afforded by the large traffic.

## 11.14 Shield (1899) on Effects of Waves on Breakwaters

In this paper to ICE, Shield starts by reminding the 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…*"[104] . The following comment "*… swell waves however … are often transformed into waves of a dangerous character*" is somewhat oblique and unclear as to whether it obeys the stated limit, or not. 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* ".

He then turns to discussing the vertical breakwater being constructed by the Admiralty at Peterhead from concrete blocks of about 40t each, laid in horizontal courses, joggled and set in Portland cement mortar. He notes that storms in winter 1896/97 had displaced (i.e. removed seaward?) 40t blocks at 5.2m and 7.2m below MLWS. Then in a storm in October 1898, waves exceeded ($H_{max}$?) 9m, displacing blocks down at 11m below MLWS, and a (bonded?) section of blockwork weighing 3300t and presenting a face of 10m x 10.3m in a local depth of 16.5m below MLWS (18.3m below MWL), slid by 50mm at a level of 3.2m below MLWS. Shield measured the coefficient of friction for similar blocks on concrete, $\mu$=0.7, so calculates an outward displacing force corresponding to about 9kPa. He then calculates indicative (theoretical velocities) for the incident wave of about 11-14m/s horizontally, 3-7.5m/s vertically.[105]

Shield then discusses wave action at a vertical quay wall adjoining the breakwater with an approaching bed slope of order 1:10. He notes waves of 1-1.5m undulating against the wall when the local depths exceeded 2-2.5m. "*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, Columbo (*sic*) and the Tyne, and notes 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, the tidal range is 5.2m. At 300m from shore, and a bed depth of 14m below MLWS, the mound was not stable even at a slope of 1:6.5 when at 1m below MLWS, the foundation being withdrawn from the wall leading to breaching.

[104] Implying that the effects of steep bed slopes, and (perhaps) wave period on wave breaking limits were little appreciated
[105] These velocities of course take no account of wave breaking over the approach slope in front of the wall.

# 12 SITE SPECIFIC REVIEW

## 12.1 Introduction

This Chapter presents historical views of selected breakwaters in more detail than in the Geographic review in Appendix D. The reviews have been presented in order of the date of construction, or of the document. There is some duplication for the more 'popular' structures. Some of those have been discussed at length in Chapter 11 where the information may be of more general relevance, *e.g.* Blyth discussed by Scott (1858) and Alderney and Dover discussed by Vernon-Harcourt (1885).

## 12.2 Tangier (1663-1684) by Routh (1912)

This breakwater was required to provide a safe anchorage to an outpost on the North African coast seaward of the Strait of Gibraltar, so commanding the entrance to the Mediterranean. It's "*value as a naval station …. was nothing unless ships could ride (at anchor) in the bay without fear of storms…'.* Being subject to Atlantic swells from the west, and severe (*Levantine*) winds from the east, a breakwater was essential, and was also difficult to build. Routh reports that "*… the building of the Mole, the greatest engineering work till then attempted by Englishmen.*"

Shortly after Tangier itself had been occupied by British forces in 1662, Lord Sandwich surveyed the bay to find the best position for a Mole to protect vessels against storms from the Atlantic (westerlies) and from 'strong Levant winds' (easterlies). Four months after establishment of the Admiralty's Tangier committee in London, the contract to build the Mole was awarded to Lord Rutherford and Sir John Lawson at 17.1s/m$^3$. Mr Hugh Cholmley was appointed as resident engineer 1663-1674, having previous experience of construction of a pier at Whitby.

Construction of the Mole started in June 1663, with the breakwater foundation started in August 1663, from York Castle 366m ENE, then 183m ESE, 27m across at the foundation. The construction targets were 15,300m$^3$ by end June 1664, then yearly 23,000m$^3$. The Mole was to be kept in repair for 5 years from completion at £6,000 / annum.

By January 1665 delays to the works (including to payments) were such that only 8,100m$^3$ had been placed. The contract price was increased to 22.4s/m$^3$ (31% increase), and progress improved such that a battery of guns was placed on it. Supervision by Major Taylor and Mr Cholmley (assisted by Henry Shere) was interfered with by Colonel Norwood (Lieutenant General in 1666) "*intermeddling in the worke of the Mole*" who favoured onward progress to intermittent strengthening and consolidation. Despite the interference, progress on the Mole reached 350m by August 1668, as surveyed by Earl of Sandwich.

By 1669, Sir John Lawson had been killed in the Dutch war, Hugh Cholmley was the sole contractor left, so the contract was cancelled. In August 1669, Sir Hugh Cholmley was appointed as Surveyor General, and the contract now appears to have been conducted by salaried Government servants to a model approved by Christopher Wren and Jonas Moore. The intended salaries were:

| | |
|---|---|
| Surveyor General, Sir Hugh Cholmley | £1500/yr |
| Deputy Surveyor, Major Taylor | £500/yr |
| Clerk Examiner, Henry Shere | £250/yr |
| Junior staff | £50-100/yr |

By April 1670 when Cholmley returned from England, the Mole had suffered serious damage in two winters, and doubts had been raised as to the wisdom of the current / intended construction. Henry Shere espoused a technique used at Genoa in which large wooden chests (caissons) were filled with stones and cementitious material were sunk onto a foundation mound of stones and rubble. This approach, discussed at Tangier with visiting Genoese engineers in 1663, had been rejected by Cholmley as too difficult to make and install the caissons in a 9ft tide and the stormier exposure of Tangier Bay. He also identified potential attack by worm as contributing to potential failure, even if installed. Cholmley's preferred construction used a mound of loose stones up to low water as a foundation, surmounted by '*great stones cemented with lime and tarrace and cramped with lead and iron'*. This work was to be protected by three rows of timber piles in : - : - pattern that supported the toe of the foundation. But, once destroyed by worm, these were replaced by stone pillars formed by 2-4 ton stones laid in tarrace and bound by iron and lead solder – although how that was performed underwater is by no means clear!

The extent of the Mole was surveyed on 19 April 1670, and found to be 400 yards long. Cholmley advised that only 73-91m further were needed, at 18m / annum, with a 55m return (to ESE?) to defend against eastern seas (Levantine winds). Cholmley proposed to continue the current construction, but failed to convince the Governor, Lord Middleton, who backed Henry Shere's suggestion.

At length, Cholmley agreed to try Shere's plan, and an attempt was made in September 1670 to place a first caisson. After significant difficulties, it was installed, but back-to-front! A second caisson appears to have been installed later in 1670 or early in 1671. Progress continued through 1671-1674, but the winter of 1674/75 caused much damage to the Mole. Cholmely was much frustrated by restrictions and lack of payments from the Admiralty. In June 1675, he submitted a proposal to a proposal for £30,000 a year, to be paid quarterly, so that he could repair and extend the Mole to 460m with a 91m SE return. He offered to return payments if the breakwater failed inspection, and offered to maintain it for £2,000 a year.

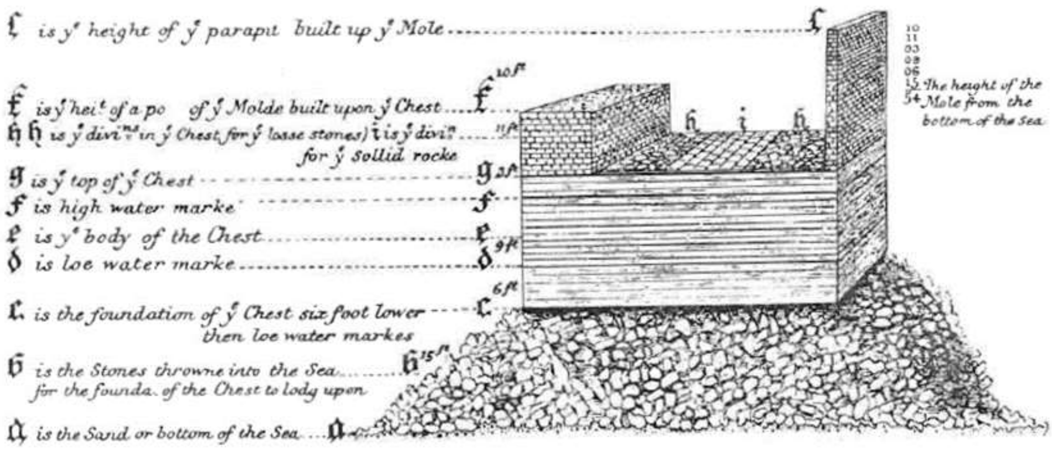


**Figure 12.1 The Great Chest constructed by Mr Shere, June 1677**
*After Routh (1912)*

Cholmley's offer to continue the work was rejected in favour of the counter-proposal by Henry Shere who offered to repair and extend the breakwater using 'great upright chests' (Figure 12.1) at £10,000 less than Sir Hugh's proposal. Henry Shere took over in June 1676, by which time it was reported that "*the Mole is in its design the greatest and most noble Undertaking in the world, it is a very pleasant thing to look on …. now near 470 yards"* (~430m*)*.

Shere placed his first caisson in September 1676 despite a strong east (Levant) wind, and rough sea, and a second was launched in October 1676. Several further were placed in summer 1677. By 22 October 1677, Shere had placed 11 chests and the Mole reached 418m by November 1677. [NB: Each chest was named, as if a ship, including: *Anglesey, Peterburgh, Craven, Coventry, Charles, York, Old Chest*. ]

As his predecessor, Shere was also substantially troubled by irregularity of payments, and was warned by Pepys not to buy materials in advance lest he be unable to pay his workforce. But not only were payments under threat, but the whole construction enterprise was attacked in 1678-80 by the Moor who attacked the quarries, then laid siege to the town, diverting staff and materials to the fortifications. Peace was concluded in 1680, but by February 1681 abandoning Tangier was threatened, finally decided in an official report in 1683. Contributing to the recommendation to destroy the harbour were:

1. Depths in the harbour had been reduced by sand being washed into the bay, and from attrition to the stone forming the Mole itself;
2. It appeared impossible to prevent siltation of the harbour;
3. The ground within the Mole was full of rocks upon which (mooring?) cables were often cut;
4. The harbour was overlooked by sandhills from which the Moors could fire cannons;
5. The proposed return would make it difficult to leave the harbour under easterly winds;
6. Various Admirals preferred Gibraltar;
7. Seas would be expected to "*beat down the Mole*" if continued into deeper water;
8. The local water supply was insufficient for even two or three ships, and was "*bad and pernicious to men's health*".

Routh argues that many of these assertions were inaccurate, but that Shere told Pepys that "*he was able to answer them all*". Certainly many of the more independently minded naval captains were in favour of retaining the harbour, but in vain as demolition of the breakwater continued through 1683 to February 1684. It had been expected that Cholmley's early construction would be easier to demolish than Shere's later work, but the contrary was true, with much more effort being required to demolish the wall over the inner sections, presumably because the slow-setting tarras and lime mortars had gained strength over the intervening years. The demolition arisings were thrown into the harbour specifically to hinder any later use of the harbour area.

Routh also draws significantly on Pepys' diaries which separately give background to the construction of the Mole and harbour, and their demolition.

Demolition of the Mole and the town took three months through the winter 1683-84, and required some 2000 men!

## 12.3 Alderney and Guernsey (1855-1860) by Lyster (~1860)

This small hand-written notebook describes observations at various harbours, chiefly Alderney, dating from 1855 and on. Lyster starts by describing on works at Guernsey with priced Bill of Quantities for work at St Sampson, and recipes for Aberthaw and Blue Lias limes. He notes use of Guernsey Granite for paving and discusses the construction of the crown wall on Alderney breakwater, use of staging, fabrication and handling of concrete blocks, commenting that a given size of concrete block cost only half of the same size granite block.

## 12.4 Alderney by Vernon-Harcourt (1873)

This paper to an ICE meeting, and the following discussion, notes that the attention of the Government was "*directed in 1842 to the defenceless state of the Channel Islands, and to the necessity for harbour accommodation*." James Walker (past-President of ICE) was asked in 1844 by the Admiralty *"to examine and report on the proper sites, and on the costs of, the proposed harbours*". Resulting from those reports, harbour breakwaters were constructed at both St Catherine's Bay on Jersey, and Braye Bay on Alderney, both to remarkably similar designs, remarkably given the dramatically different exposures of the two sites. Later noted that: "*The force of the sea during storms in this part of the channel … was not understood at the commencement of the works*".

The original intention at Alderney was to construct a small harbour in Braye Bay on the north west side of Alderney.[106] Two breakwaters were envisaged commencing at Grosnez (the western) and Roselle Points respectively. Successive Boards of the Admiralty then enlarged the notional harbour area, but they failed to commence the eastern breakwater.

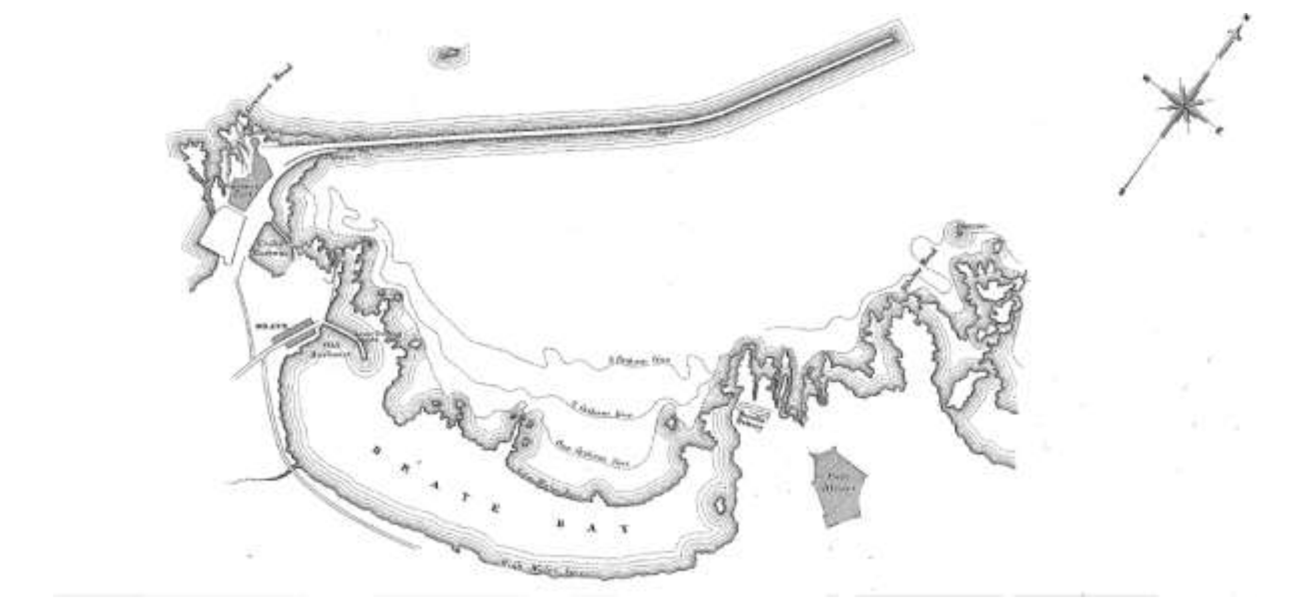

**Figure 12.2 Admiralty breakwater in Braye Bay, Alderney**
*From Vernon-Harcourt (1873)*

The design section (generally used over the first 410ft, 125m, length) used a large rubble mound surmounted by two separate walls being filled by loose rubble. The walls were founded at LWOST. The 14ft (4.3m) thick seaward wall was formed by large undressed stones (one assumes rough hewn to shape), laid as closely as possible. No cement (possibly meaning lime mortar) was used for the wall up to +21ft (6.4m) LWOST. The harbour side wall was founded at the same level, but only 12ft (3.7m) thick. Both walls were battered, the seaward by 0.75H to 1V, the harbour-side by 0.33H to 1V. These walls were surmounted by a lower quay on the seaward side, paved by granite, and by a near vertical seawall topped by a 4ft x 4ft (1.2m) parapet wall on the seaward face, and an upper paved promenade.

Vernon-Harcourt argues that the "abundance of stone on the island" supported use of the 'pierres perdu' system for the foundation, and argued that the Mannez stone used was a hard sandstone or quartzite, "*suitable for any work in which strength … is a desideratum*".[107]

Construction to this design commenced in 1847 and continued to the spring of 1849, by which time experience over two winters had already shown up significant weakness in the design. The rubble mound had been disturbed and washed into the harbour, and considerable damage had been done to the walls. The design section was amended after 410ft (125m) reducing the batter (steepening the face), masonry was set in Medina cement, and the seaward face of the wall started lower. The foundation level had indeed already been lowered as far as practicable without divers after the first 150ft (46m) length. Having used end-tipping hitherto, the new lower mound level required use of hopper barges, which in turn required shelter, afforded by forming a small construction harbour sheltered by piers of 250ft and 100ft (76m and 30m) respectively.

In the new works, it was found that the rubble mound was not disturbed at elevations lower than about -12ft (-3.7m) LWOST.[108] Work to the revised design proceeded "*as soon as diving apparatus and the hopper barges were procured*".

[106]The site was chosen for military reasons, to limit the ease of scrutiny from the cliffs of the Cherbourg peninsular, see Allsop (2020).
[107] We will see later however the abrasive consequences of dumping Mannez stone onto the upper slopes of the mound at this exposure.
[108] Note: this is in the absence of the reflecting superstructure!

The revised seaward wall from 410ft (125m) from the root was founded at -12ft LWOST to a width of 21ft (6.4m), but with a vertical rear face. Granite facing was used below LWOST, backed by Portland cement concrete blocks. Mannex stone behind the wall face was now set in Medina cement.[109] The lower wall batter was reduced to 0.5H to 1V. The upper promenade continued, gradually reducing its batter upward, shedding its small upper parapet wall after a short length. This design persisted to about 2700ft (823m) from the root, reached in 1856. By now the Admiralty had altered priorities, so the engineers redesigned the layout, and the breakwater section, see Figure 12.3

The line of the breakwater was continued to 2900ft from the root (884m). After this, the line was curved outward over 520ft (158m), then continued straight heading north-east (perhaps for 1780ft (543m)[110] An outer head was built in 1864 using a foundation set even lower, the final 42ft (12.8m) being set at -24ft (7.3m) LWOST. By now, significant iron bars and diagonal straps were used to connect blocks around the (square) end.

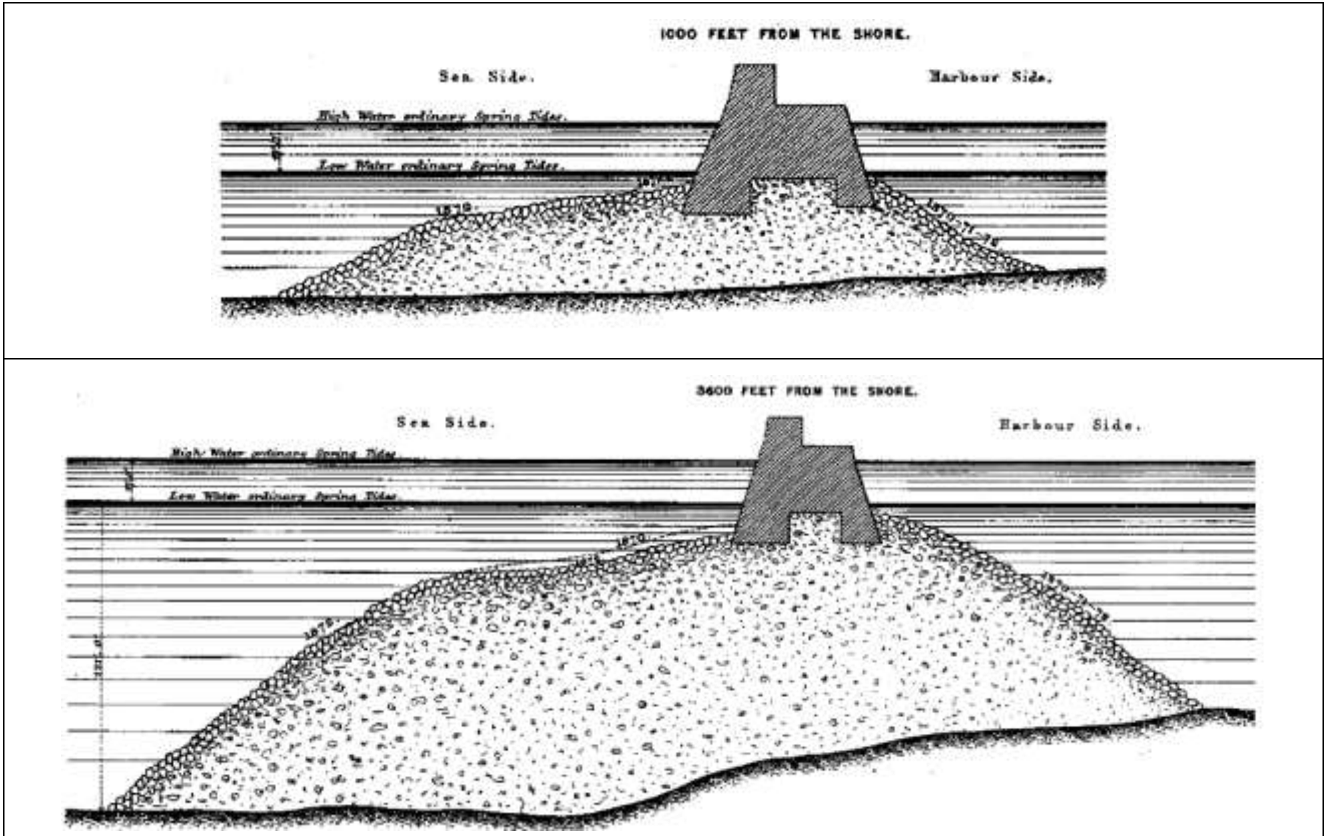


**Figure 12.3 Sections of Alderney breakwater**
*From Vernon-Harcourt (1873)*

The breakwater had not been free of damage during its construction[111], but Vernon-Harcourt claims that most damage had been at points where the mound crossed / intercepted rock outcrops, where the superstructure might cross from movable mound to fixed outcrop. Other instances of damage were relatively minor. A storm in January 1865 however forced two breaches, both completely through the superstructure, over widths of 50ft (15m) and 130ft (40m). Another breach occurred in January 1866, a smaller one in February 1867, and another 60ft (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 and Sir A Clarke "*were requested by the Board of Trade to visit Alderney and to report on the best measures for securing permanently, either the*

[109] It may be noted that the use of Portland cement mortars or concrete blocks increased steadily through the construction period (1847-1870).
[110] It is not entirely clear in the paper if this includes some length for the outer end detail.
[111] Perhaps an under-statement!

*whole or the inner portion of the breakwater*." They visited in May 1870 and suggested the removal of the promenade wall, deposition of a large additional foreshore of rubble or concrete blocks, but before that the current rubble mound should be monitored to identify changes produced by the sea.

Damage, and repairs, continued to July 1872 when Parliament decided to cease funding repairs. By then the total expenditure over 25 years had been £1,274,200, of which £57,200 (4.5%) had been for repairs. Maintenance / repairs were abandoned in 1872 and 1873, but damage continued through that winter with another breach at the weak point near 4480ft (1366m).

Vernon-Harcourt continues to try to analyse reasons for the damage, although this is hampered by the lack of any data on waves, loads, or mound movement. Most points of damage, aside from those at the rock outcrop edges, were seaward of 3400ft (1036m) from the root, where the sea bed was lower than -90ft (-27.3m) LWOST, indeed reaching 130ft at the head (40m). The paper notes that these depths are significantly deeper than at Portland, Holyhead, Dover, Plymouth and Cherbourg. The author notes also the rapid tides, and exposure to the Atlantic. He also notes that mound settlement reached about 1/20 of the mound height even after time was left for consolidation.

Particular weaknesses occurred above the outcrop points, at junctions between each fortnight's work, and then again at season joints. Cracks opened in the face and interior allowing air pressures "*greatly aided in producing breaches*". He speculates that a lower superstructure with less batter, and a lower foundation level would have been more effective, but without explicitly identifying how. He also notes that the original construction methodology was based on a smaller harbour, in relatively shallow water, and without a knowledge of the exposure.

Considering the mound, the paper records that about 4,000,000 tons, of which 332,000t over seven years was considered as maintenance. [112] Soundings suggested that the mound tended to a 1:7 slope on the upper part (instead of 1:5), changing at about -20ft LWOST, -6.1m (not 4.6m, 15ft as assumed previously. He noted that it was not possible to maintain the mound at LW by dumping rock.

## Discussion to paper 1347, pp 84-102

**Mr Redman** argued that "*...many of the sweeping criticisms upon the breakwater had been made without all conditions surrounding the work having been taken into account.*" He credited two people further to those acknowledged by LFVH: Major Newsome, Royal Engineers who was stationed at Alderney, and the late Mr James Cooper MICE who "*... prepared the specifications and working drawings and taken the general supervision of the works ...*" He noted that, similarly to Holyhead, at Alderney "*... the harbour, projected originally for a smaller enclosure, was subsequently diverted from the shore. Had a curve of contrary flexure been adopted, there would have been considerable addition to the area.*" Vernon-Harcourt had remarked that the St Catherine's section was similar to Alderney, but Redman contended that the similarity was with the pier at St Helier. He then blamed damage on the large mound towards the outer end of Alderney as "*... unequal settlement caused the longitudinal and transverse fissures which had proved to be points of weakness, and ultimately led to the breaches.*" He then contrasts the exposure of many different breakwaters by their "*offing*", presumably maximum fetch:[113]

| | |
|---|---|
| Alderney | 3000-4000 miles to W and SW |
| Dover | 150 miles SW to SW by W |
| Cherbourg | 100 miles |

[112] 1 ton of Mannez stone occupied 20ft3 (0.57m$^3$) on the mound, 13.5 ft$^3$ (solid?) weighed 1t, so the placed porosity was about 33%, not unreasonable given the (probably) wide-graded nature of the dumped stone.
[113] This analysis of comparative exposure is however confused by citing several different depths at different distances from the breakwaters.

| | |
|---|---|
| Portland | 140 miles SW to SSW |
| Plymouth | width of the Atlantic |
| Holyhead | 100 miles NW and N by E |
| Wick | 600 miles N and NE |

On Alderney, Redman noted that "*... it was a remarkable fact that the pierhead, in the most exposed situation, and on the greatest altitude of rubble mound, had remained uninjured from the time of its formation. He had no hesitation in expressing the belief that the work might be maintained in a safe and economical manner. He did not recommend removal of any portion of the work, or any extreme measure.*" [There would appear to have been proposals to remove the parapet over some length, and/or to abandon the outer end – see later.]

**Mr John Jackson** had been the contractors' agent 1857-1866, and had worked on the fortifications of Alderney 1852-1857. He asserted the Mannez sandstone to be "*much harder than granite ... harder than the Guernsey granite used in the streets of London*." The quarrying used 74t of powder per year from a face 23m high. The stone was moved by 4 locomotives and 400 wagons, with a maximum of 3000t tipped in a day. "*For many years it was stipulated that every barge load should be supplemented by ten wagon loads of sand and shingle.*" [114] This process was later discontinued[115]. He then described the erection of staging at the recommencement of each year's work, using a helmeted diver to excavate holes in the previously-dumped material to receive the support piles. During construction, six divers were in operation at any one time, four on the sea-side, and two on the harbour side, working in four hour shifts, three shifts per day. The divers' working stage (hanging 3m below the main stage) was removed each night. The main stage at 6.1m above HW was "*seldom disturbed*". He notes however the particular behaviour of "*… summer ground seas ... were very dangerous ... and some of the men lost their lives through them. They came on with scarcely any warning… In a few minutes a tremendous body of water would be flung against the wall …the wave would mount up a vast height into the air.*" [116] The effect of the reducing mound level along the breakwater then becomes clearer: " *... these seas were not so heavy during the last few years of the construction of the breakwater ...*"

Jackson discussed the operation of delivering blocks to the divers, and then to the masons once the blockwork emerged above LW. Medina cement mortar (1 part cement to 2 parts sand) was used to bed the blocks. Medina cement being brought fresh from the Isle of Wight so that its setting was not impaired.

He noted that repairs amounted to less than 4% of the capital cost, and confessed surprise that the maintenance / repair costs " *... should have been the subject of deliberation by a committee of the House of Lords.*" 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…*".[117]

**Sir John Coode** joined in the criticism of the shape of the harbour, being too narrow, yet with an entrance too wide and far too open to waves from the North-East. He also noted that, whilst "*... the exposure was great ... the very heavy sea against the pier was mainly due to the peculiar configuration of the bank ...*" he noted similar effects at Eddystone "*... at the close of a gale…*" i.e. when wave periods had lengthened. He noted that the Mannez stone had a density of 13.5 $ft^3/t$ ($2.44t/m^3$), but argued for the addition of sand to the mound to increase its "*solidity*".

**Sir John Hawkshaw**'s connection with Alderney arose from his commission (with Colonel Sir Andrew Clarke) to advise the Government on some method of repairing or improving the breakwater to reduce or prevent "...

[114] Note inconsistency of units of volume!
[115] No date was given.
[116] This sounds very like the Chesil beach overtopping of 1979 when a 'packet' of swell waves inundated Chiswell at the Portland end of Chesil Beach.]
[117] So much for the non-military pretence of the 'harbours of refuge'.

*the constantly recurring mishaps.*" He had suggested that cross-sections should be measured "... *to ascertain whether the mound was stationary or shifting and ... to what extent.*" He had suggested that this "…*would take a couple of years* … " before he could advise "... *what steps ought to be taken.*" Yet the Government decided "… *not to carry on the works*…" before that period had elapsed.[118] Sir John then described his thoughts on the occurrence of breaching, having inspected breaches from 45' wide (13.7m). The shape of the mound caused seas to be thrown against the wall "*with great violence… in falling back again their momentum drew away the deposit from the base of the wall, and the bottom courses dropped down, having nothing to stand on.*". The (uncemented?) rubble hearting was then "*fetched out*" with erosion working through to the harbour side wall, thus causing "… *breaches clean through*".

**Colonel Jervois** (Royal Engineers) commented on the military need for the harbour. He noted that the harbour had been proposed by Admiral Sir Edward Belcher when use of steam propulsion "... *was in its infancy*". At that point the suggested location was on the South-East side of the island at Longy Bay. Mr Walker had however proposed "... *the smaller harbour at Braye Bay, which had gradually grown to its present dimensions.*" He hazarded that if asked now whether to construct such a harbour he would reply "No", but having got it, if asked whether to maintain it he would reply "Yes", indeed he concurred with Sir John Coode on the need for adding the eastern arm.

**Sir Edward Belcher** explained that he had been summoned by Government in August 1842 "... *on secret service* ..." to examine 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…"* [at Braye Bay] *"... would render it utterly impossible for any disabled vessel to get in…*". He had then suggested locating the harbour at Longy, protecting the harbour by "*damming*" by Isle du Rez. He felt it would also have been possible to cut through to Bray Bay to allow a westerly exit. His advice to the Admiralty in September 1842 was that a harbour at Longy would cost £1,500,000. He had since given advice to the Earl of Auckland who believed that "... *nothing satisfactory could be constructed* ..." at Braye. In 1852, Sir Francis Baring had summoned him to the Admiralty to be told that "... *the former Commission was still in force ... ordered to go to Alderney harbour and report upon it."* They were instructed *"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 worth while to expend £600,000 more on the eastern arm.*" Mr Walker was also instructed to go "... *in order that he might be there in a gale of wind.*" It appears however that both Walker and Belcher advised against the eastern arm, perhaps convinced that the concomitant concentration of tidal flows would scour the foundations. Belcher concluded his contribution to the discussion with the somewhat 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 an extensive response **Vernon-Harcourt** noted that the proposal for the eastern breakwater had not been abandoned until 1862. The estimated cost in 1852 had been £137,000, for the "*most extended design*" £300,000, and the "*last design*" £100,000. 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*".

**Vernon-Harcourt** then sketched a section of the wall to illustrate his assertion that the superstructure had settled rather more on the harbour side than at the seaward. He suggested that 25,000t of large stones needed to be dumped onto the foreshore each year, at a cost of some £3,000 / year. Adding a cost of repair of the wall

[118] So much for evidence-led decision-making!

estimated by averaging the costs over 1864-1872 at £5,500 per year gave a projected annual cost of ~ £8,000/annum. He then summarised the maintenance costs by section along the breakwater:

| **Distance from shore (m)** | **Completion date** | **Maintenance cost (£)** | **Period of maintenance (yrs)** | **Average annual cost (£)** | **Cost per m (£/m) †** |
|---|---|---|---|---|---|
| **0 – 305** | 1852 | £1,850 | 20 | £93 | £0.3 |
| **305 – 610** | 1855 | £2,550 | 17 | £150 | £0.5 |
| **610 – 915** | 1860 | - | 12 | - | - |
| **915 - 1220** | 1862 | £17,160 | 10 | £1,716 | £5.6 |
| **1220 - 1430** | 1864 | £24,340 | 8 | £3,042 | £14.5 |

* *All costs at 1873* † *This column added.*

He noted that these costs tended to support the suggestion of retaining the inner portion (say 1010m), but abandoning the outer sections if Government considered that the maintenance costs were beyond the value of the harbour. He calculated annual maintenance costs for the inner 1010m might be equivalent to £2,900 (£2.9/m.year). If the full length were to be retained, then he suggested that the foreshore should be protected by large concrete blocks.

Further discussions after the reading of the paper were tabled through the Secretary. [119]

**Mr J A McConnochie** had been involved in designs and estimates for the works 1855-1868. He noted that the outer end, constructed in 40m of water (at LW) "*… had stood unmoved on the rubble mound, and was likely to remain so.*" He noted that "*... all the old plant had been disposed of, and new plant would be required…*" perhaps with a capacity for 30-40t blocks. He noted that breaches "*... never extended down to the bottom of the walls, the two lower diving courses being generally left uninjured ...*" There had been little damage to the inner section, some 3,400ft (1040m) from the shore.

**Mr Parkes** had acted as Resident Engineer on these works for Walker & Burgess in 1848-49, "*... and he looked back on those two years as the most instructive and important of his professional life ...*" particularly when the "*… inadequacy of the originally designed section to withstand the great force of sea became apparent.*" He contended that a major contribution to the damage had been omitted from the paper – the difficulty in maintaining the unfinished end "*... even during ordinary gales…*" as he believed that all the damage (during construction?) started from the exposed end. He then turned to the depth at which the wall foundation layers had been set. He found a discrepancy between remarks by Mr McConnochie: "*... the lower courses had never been disturbed…*" and Sir John Hawkshaw: "*...the lower courses, having nothing to stand on, dropped down ...*".[120] Parkes then blamed the form of the superstructure rather than the depth of foundation, although he then acknowledges the difficulty of maintaining continuity " *... on a yielding foundation… Had the superstructure ... consisted of a succession of detached masses, each mass might have sunk without causing dislocation.*" He therefore blamed the "*rigid continuity of the work*", although he felt that this problem would reduce as settlement of the mound reduced in time. He was pleased that "*... the idea of great additions to the foreshore had been for the time abandoned ...*".

**Mr Bindon Stoney** remarked that the paper "*... had faithfully recorded the inherent sources of failure ... first, the insufficient depth below low water at which the base of the superstructure was laid, and secondly the subsidence of the rubble mound which caused cracks and fissures in the continuous masonry above ...*" He then speculated on the possibility of using large (say 350t) concrete blocks as at Dublin, [although Stoney does

[119] It is probable therefore that these contributions were based solely on the written paper, not arising from any discussion at the meeting.
[120] Parkes may have missed a subtle point here in that the lower courses might indeed have 'appeared' to be undisturbed, those layers above having been washed away, whilst they might well have moved enough to 'un-bond' the blocks above, see particularly comments by Scott (1858) and the work many years later by Marth et al (2004, 2005), discussed in 3.5.]

not seem to have acknowledged the major difficulties in transporting and placing such blocks in the currents and waves prevalent at Alderney.]

**Mr T R Winder** concluded that the (old) 12ft (3.7m) rule should be revised and foundations for (vertical) walls should be founded at 20-24ft (6.1-7.3m) below Spring Low Water. He supported this by a [slightly tortuous and unclear] analysis of experience at St Ives where foundation material had been moved at 22-26ft (6.7-8m) below Spring LW.

In closing the (written) discussion **Vernon-Harcourt** commented that both Mr McConnochie and Sir John Hawkshaw were probably correct [as discussed above]. But he also noted that the foreshore had actually lowered "*in one or two places*" to below the level of the superstructure foundation in the last 2-3 years.

## 12.5 Aberdeen by Cay (1874)

The new South Breakwater at Aberdeen was completed in autumn of 1873 (paper 1389 was read to ICE on 15 December 1874). This breakwater preceded the proposed 3rd section (extension) of the North Breakwater. It appears to have been the first use of large concrete-filled bags to form the foundation layers, followed by concrete blocks, 9-24t each, then topped by a mass-concrete upper section.

For the foundations, bags (initially) of 5.25t of concrete, and (later) of 16t, were placed from bottom-opening iron skips. These were lowered under the supervision of divers, the bag being discharged when the skip is over the required position. The filling concrete was 3.5 parts shingle, 2.5 of sand and 1 of Portland cement. Construction staging used 68 Oregon pine masts or piles, each 20m long in the finished work. [Cay devotes some 3 pages to discussing the composition and materials of the staging.] Travelling carriages carried 25t steam Goliaths by Stothert & Pitt. A 3t steam derrick crane by Butters Brothers of Glasgow was used to erect staging. A 15t hand-powered Goliath was also used. Cay comments on the steam Goliaths that "*the economy in working it more than paid for the additional first cost of the steam power*".

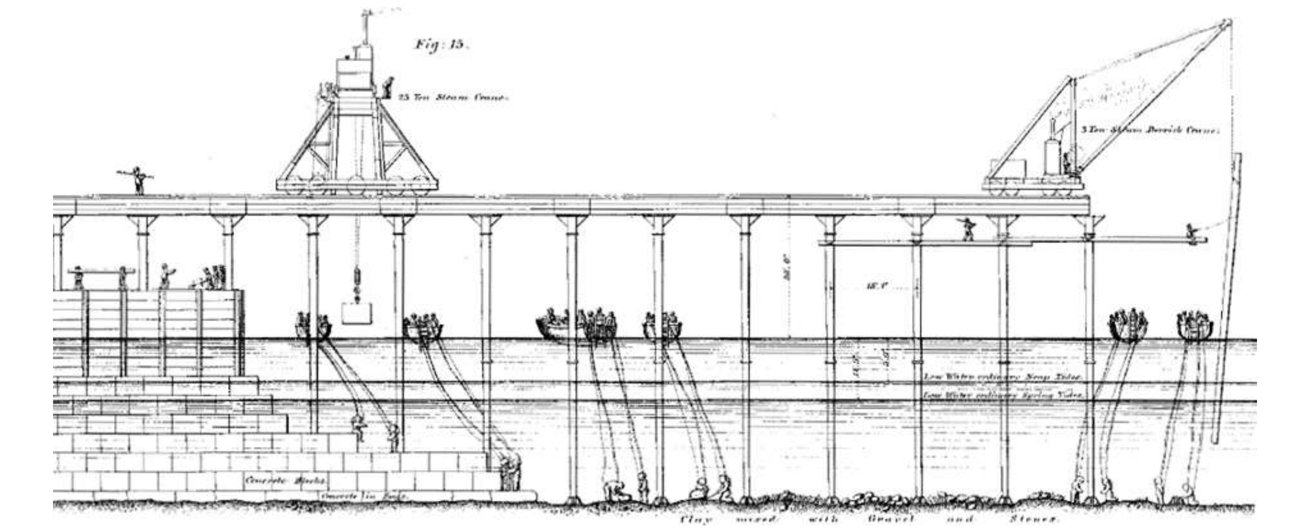


**Figure 12.4 Construction activities showing diver operations at Aberdeen**
*From Cay (1874)*

In the first year of construction, mass-concrete bay lengths varied around 5-6 m, extended in the second year to 5-9m long, the increase being achieved by including large concrete 'plums' (up to 25%) in the in-situ concrete. The formwork (generally?) used sacrificial iron tie rods across the construction. Concrete mixing used four Messent machines, each mixing $0.4m^3$ at a time, and delivering $9m^3$ per hour.

Around the toe of the wall, large (100t) bags of concrete were dropped to form toe protection from a pine hopper box suspended from brackets mounted on the upper wall.[121] Diving support used diving helmets by Siebe & Gorman with six air pumps supplying twelve divers, example shown in Figure 12.4. Thirty two divers were used, of whom thirty were trained on the project.

### Discussion to the paper

**Mr Parkes** indicated some doubts as to whether the approach at Aberdeen would prove to be economical elsewhere. He was however impressed by the use of the large concrete-filled bags. He then discussed the importance of allowing for settlement by NOT bonding blockwork. He favoured unlinked columns of blocks to allow differential settlement.

**Mr Grant** agreed with Cay that substituting concrete in large bags for the lower blockwork could have saved much expenditure on staging and cranes, and making blocks. He favoured however stepping the working end rather than vertical joints, pointing to work on the Thames Embankments where the working face was stepped at 1V to 3H.

**GR Stephenson** commented on the form of harbour entrances, fearing that "*little or no attention was paid to the position of the piers … admit … vessels running safely in, seeing that steam-tugs were always available to tow vessels out*".

**Sir John Hawkshaw** gave a spirited defence of Mr Cay's work, denying that harbour designers ignore the needs of mariners in designing harbour entrances. He also countered other points on breakwater width ("*in many seas, a breakwater 12 ft wide would be of no more use than a sheet of paper*"), on forms of construction, and the use of large concrete blocks.

The discussion continued with an extended debate on the apportionment of costs of temporary works and plant attesting to the merits (or otherwise) of the approach adopted for the South Pier.

**Cay** then continued to highlight his intentions for the planned extension of Aberdeen North Pier. The use of large concrete-filled bags would be continued, now larger at 50t each (although he also describes the relative costs of hopper barges for 50t and 100t bags). The entire upper part of the structure would then be cast by in-situ concrete in frames (formwork).

**Sir John Coode** had used Portland cement concrete for about 10 years in both facing and backing fill, and in blockwork (so since 1864). He noted the previous use of (small) concrete-filled bags deposited in iron skips by James Barton at Greenore, Carlingford Lough.

**BB Stoney** commended the use of large concrete-filled bags, particularly in contrast to 10-20t stone blocks, relatively easy to displace if the foundation on which they sat were to be at all soft. He had used concrete-filled sacks with success six years previously (so 1868). He mentioned briefly the use of 300-500t concrete blocks being employed in Dublin harbour. He also recommended including a Δ tube across between the forms, allowing the tie rods to be withdrawn and re-used.

[121] Again, Cay gives significant construction detail of the hopper box.]

## 12.6 Whitehaven by Williams (1878)

Williams is mostly concerned with the history of the harbour developments, and the (frequent?) disagreements between client and designer. Few details are given of the breakwater construction, rather more on arguments between users / sponsors and the designers. The later part of the paper describes ancillary features of the docks, so not covered here.

An initial harbour was inspected by Smeaton in 1768 who suggested enlargement by extending the West breakwater (New Quay) and adding an East breakwater (termed the North Quay). As the cost of this large expansion might not be met by the harbour trade, for the short-term he suggested a simple northerly extension of the New Quay (west breakwater) Figure 12.5.

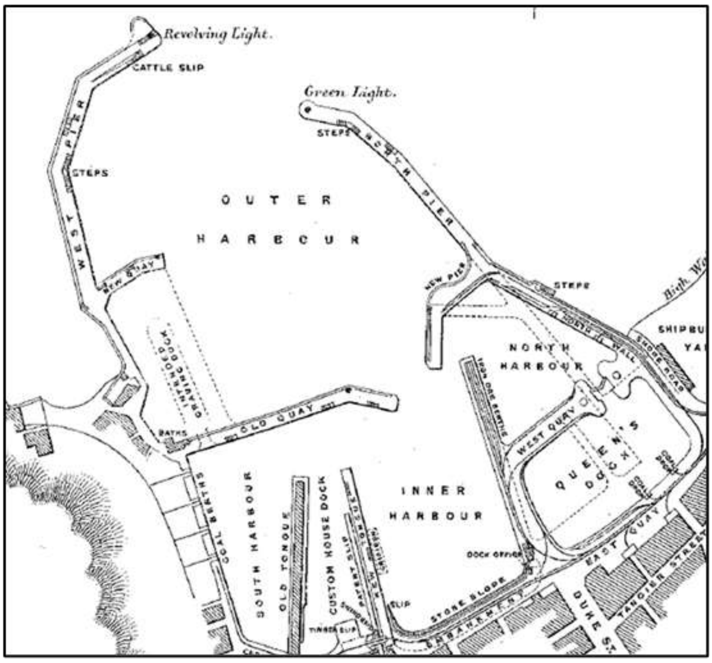


**Figure 12.5 Whitehaven Harbour, after Williams (1878)**

In 1814, John Rennie proposed a 300m extension of the West breakwater, and 180m of the East breakwater. Each would include a 30m spur jetty behind the breakwater heads. Other designs were also produced. It was not until 1824 that work commenced to a Rennie design. The extension of the west breakwater (Figure 12.6) was however accompanied by siltation as longshore currents carried sediment into the now calmer waters, and by difficulties in navigating past the extended pier.[122]

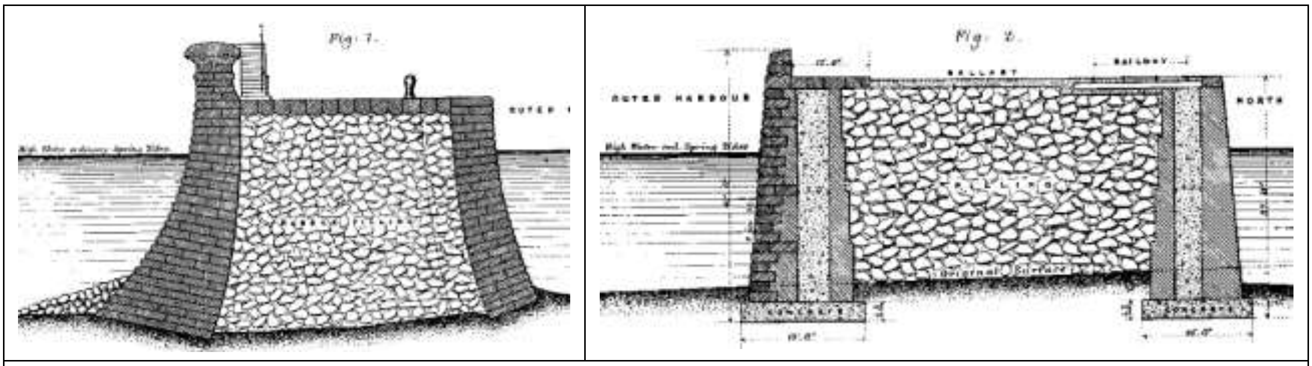


**Figure 12.6 Whitehaven west and east breakwaters, after Williams (1878)**

In 1833 (now Sir John) Rennie found sand accumulation up to 2ft (0.6m) in places, driven by the "*counter or eddy current setting to the south*". He urged the Trustees to construct the East breakwater as originally intended. This was started in 1833, but a vessel incident in a storm in 1836 caused another crisis of confidence, with a series of other 'designers' (including James Walker) being asked to propose new solutions. Despite these interventions, generally repudiated by Rennie, the end of the West breakwater was completed in 1839, and of the East breakwater in 1841.

[122] One gets the impression of users always ready to find fault.

## 12.7 Dublin Harbour by Griffith (1879)

This paper starts by summarising the history of Dublin as a port, initiated by 'cleansing' the Rivers Liffey and Dodder. The first (artificial) channel was formed in 1711 from the city to Ringsend using a bank of timber and stone on the north side, and a masonry quay wall on the south side. The south side quay was extended out to Pigeon House Fort as a piled jetty out by a length of 2992m, although progress on the timber jetty was slow. About 1748, the length from Ringsend to Pigeon House Fort was replaced by a double skin of masonry (rubble) walls, infilled by sand. This was continued out towards the Poolbeg lighthouse starting in 1761. After the formation of a dedicated Ballast Board, the Great South Wall was finished in 1796, with a small ship basin at Pigeon House.

Comparing soundings along the channel from 1711 and 1725 against a survey by Captain Bligh (later of HMS Bounty) in 1800 indicated that the improvements (reduced siltation) at a cost of £200,000 had been successful. Griffiths rejects criticism that it had not reduced depths over the outer Dublin bar, an objective that had not been aimed for. He does however discuss at length the debates on the need for the northern wall to "… *cause deeper water between the bar and Dublin…*" at a projected cost of £246,000. The issue was also considered by Rennie who suggested three modifications, but even at a cost of £252,000 would only increase depths over the sand banks by about 0.3m. Alternatively, to give a more substantial reduction of (say) 1m would require works of possible cost £656,000. Somewhat more extreme proposals included a new ship channel along the north part of the bay and through to Howth. Whilst these were debated, between 1802 to 1819, the Ballast Board invested in replacing rubble sections of the Great South Wall by ashlar walls with rubble infill. Steam dredging was introduced from 1814 with a bucket ladder projecting from the vessel side.

Following detailed surveys by Francis Giles (probably 1815-16) the Great North Wall was proposed in 1819 and started in 1820. Progress was rapid and appeared to have significant effects on the sandbanks, Telford was retained to review the state of the work and of its effects. It seemed that the inner sections of the new wall to MHWN and the outer sections to mean tide were effective in reducing sand levels, even so Telford suggested further narrowing of the entrance, accepted by the Ballast Board. The Great North Wall was completed in 1825 to a length of approximately 2750m at a cost of £95,000.

Despite the successes in maintaining the depth over the bar to 3m at LWS, further changes were proposed and reviewed. Sir William Cubitt attributed the "… *great depth and improved channel over the bar … entirely to the erection of the Great North Wall …*" Captain Washington appeared to agree in 1845, and James Walker in 1861.

## 12.8 Newhaven by Carey (1886) [123]

The harbour at Newhaven on the Sussex coast developed as a river-mouth harbour with training walls. In 1788, two wooden piers around 30m long trained the river entrance. This scheme however accumulated a shingle bank in front of the mouth, so the entrance was moved eastward. Royal Commissions on Harbours of Refuge and Tidal Harbours considered potential improvements at Newhaven and elsewhere. By 1847, the London and Brighton Railway had been extended to Newhaven, and the ferry to / from Dieppe carried the greater proportion of "*passenger and carrying trade between England and France*". Even so, the harbour was still blighted by the shingle bar! In July 1878, Fred Banister was appointed as Engineer-in-Chief to the new Newhaven Harbour and Dock Company to commence construction of a new outer harbour. This was to be protected by a new 855m western breakwater reaching out to -5.5mLW, mean spring tides rising by 6.1m. He intended to found the (essentially monolithic) breakwater on large (100t) concrete bags based on the system developed by William Dyce Cay, see: Cay (1874), Vernon-Harcourt (1885), and Turner (1986). In this instance

[123] Much of this paper is devoted to overall harbour design and its origins, of relatively little interest to the present purpose.

the jute / canvas bags for the seawall were rendered water-tight by coating inside with marine glue, perhaps also those for the breakwater

A new steam-driven hopper-barge (see Figure 12.7, and Figure 12.8) was commissioned from Simmons & Co of Renfrew to hold and deposit the 100t bags. The hopper was reverse tapered (wider at bottom than top), and the doors were designed not to snag the falling bags. A new concrete batching and mixing machine was devised by Carey and Latham, constructed to ensure that 100t of concrete was fresh, well mixed, and delivered into the bag within 20 minutes. Gravel and sand for the concrete were gained from the beaches. For the main works, a further continuous mixing machine was devised to output 55m$^3$/hour. The main breakwater monoliths were cast between timber forms in 9 or 12m lengths using 5 parts shingle, 2 of sand and 1 of Portland cement.

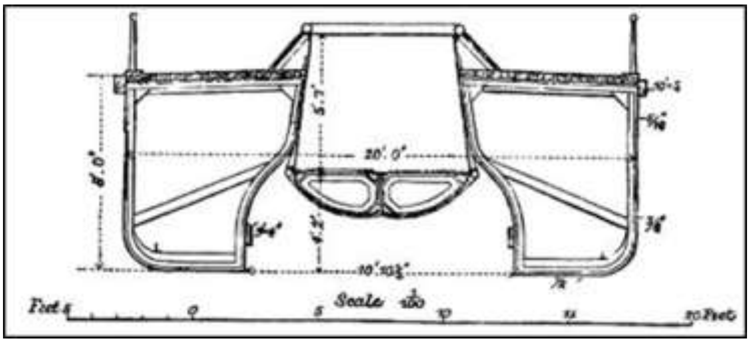

**Figure 12.7 Bottom-dumping barge for concrete–filled jute bags**
*After Carey (1886)*

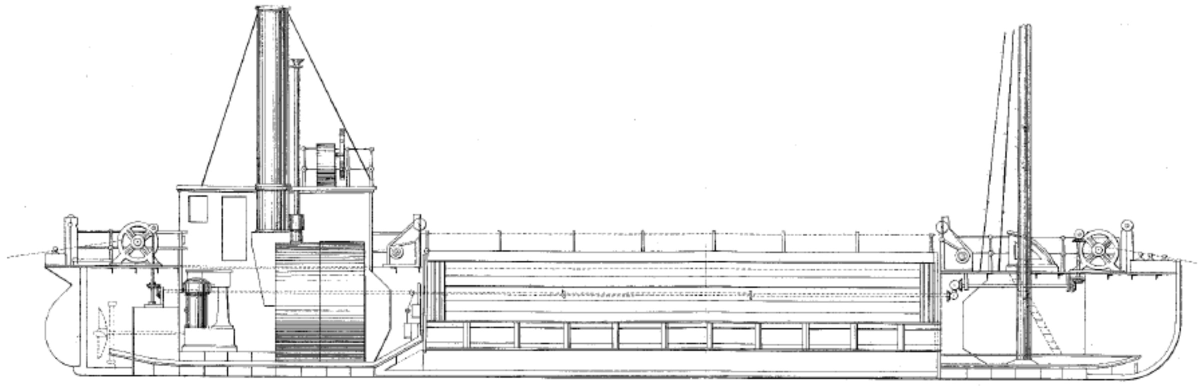

**Figure 12.8 Bottom-dumping barge for concrete–filled bags**
*After Carey (1886)*

Carey concludes the paper with various tables and a discussion on testing cement briquettes. Two pages of engravings illustrate the paper, including details of the key concrete mixing and placing plant.

## Discussion to the paper[124]

**Sir John Coode** noted that the large mechanical plant at Columbo were required to maintain progress on the works when other access would not be possible. The Titan crane by Taylor of Birkenhead, and concrete machinery and traveller by Stothert & Pitt "*had given entire satisfaction*". On wave loadings, he contended that waves were most violent during the monsoon swell, and part-way out along the breakwater rather than at the head (in deepest water). He also ascribed increased violence to reduced aeration of the water[125].

**Vernon-Harcourt** reminded the meeting that large concrete-filled bags had been used successfully by Dyce Cay at Aberdeen before they were used by Carey at Newhaven. He also commended work by Mr Strype in depositing underwater concrete from skips, allowing him sometimes to avoid use of formwork. He noted that

[124] The discussion at this meeting ranged far and wide, covering the breakwater at Columbo as well as generic matters. These notes have concentrated on matters of general utility.
[125] In part this might be true, but not at full-scale.

it was a false economy to use small proportions of cement for concrete in exposed positions. Vernon-Harcourt disagreed with Sir John on the effect of aeration, ascribing greater violence to longer wave periods.

**P.J. Messent** discussed the North and South breakwaters at the entrance to the River Tyne (Figure 12.9), constructed with concrete blocks (some faced by stone) and OPC concrete hearting. For the greater part he had avoided using rubble, the blocks being laid on a small rubble mound over sand. Large concrete toe blocks were laid along seaward and landward toes. Messent then disagreed strongly with the comparison of costs by Carey, particularly as the examples chosen differed so strongly in depth and exposure. He contended that forming the main structure in concrete-filled bags was still strongly weather sensitive, whereas building concrete blocks could take place even when on-site construction was not possible.

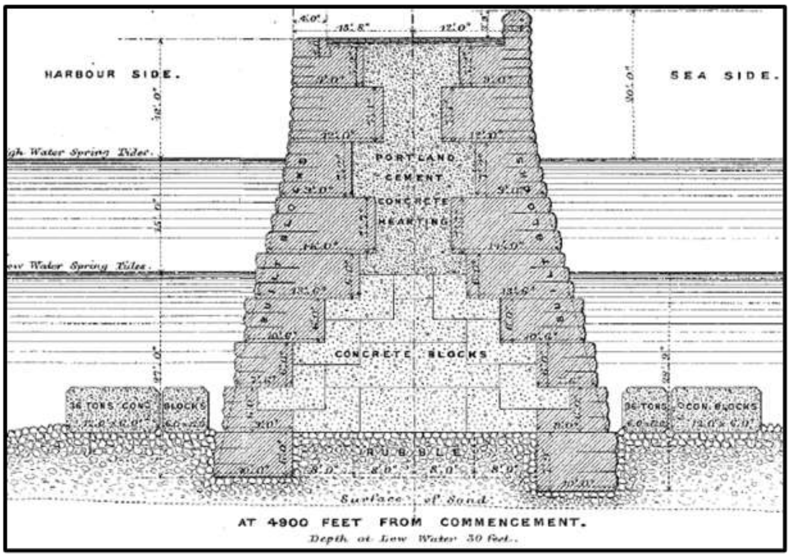


**Figure 12.9 Tyne entrance breakwater**
*After Messent in discussion to Carey (1886)*

**John Dixon** had observed construction of a mile long breakwater (1.6km) at Grand Canary out to depths of 50' (~15m) using 30t concrete blocks made with lava from the construction site, local lime to which about 2% of Portland cement was added. The cementing medium (including sand) was mixed in approximately similar volume (51%) with block lava (49%) to make blocks of 2.5t/m$^3$. Blocks were left to "*weather*" for 4-6 months.

**William Parkes** had previously introduced sloping blocks at Kurrachee, where he believed the wave action at least matched that at Columbo, 5m having been measured. He agreed on the need to change construction form as the works moved into deeper water. He then discussed an experiment in grouting shingle. He filled a wooden box (0.26m$^3$) with shingle, then added water to measure the void volume. He prepared a volume of cement to match the void volume. He then re-filled the shingle with water nearly to the top. He "*had the cement formed into grout in the ordinary manner*" - here it is entirely unclear whether he included sand (as most grout does) or not. In any case he then poured the grout into the shingle until filled to the top, using less than ½ of the mixture. On dismantling the box, he found that he had filled the voids, "*every cranny was filled by cement*". But "*it never hardened at all*". He concluded that "*to get the full strength of the cement, the minimum amount of water must be used*".

Parkes then discussed the form of dredger that was most efficient, agreeing with Carey that a stationary dredger with attendant barges was to be preferred over a hopper-dredger.

**A Giles** agreed that more cement was needed in concrete mixes for sea works, he cites 1:6 to 1:7, but he needed better advice on the time after mixing to be allowed before putting "*concrete into water to set*". He cited recommendations of 8 or 18 hours, but felt that greater clarity was needed. He then discussed the construction of dock walls in concrete advising that measures be taken to protect the wall surface against abrasion / vessel impact. He also noted a propensity of long walls to "*crack at about every 40ft*" (12.2m).

**Harrison Hayter** had previously described use of concrete blocks to form a wave-breaker at Ymuiden, and was presently constructing a similar wave breaker in advance of breakwaters at Mormugao on the west coast of India. He discussed the use of Titan cranes at Ymuiden, two of which were lost when the works then under construction collapsed under storm attack. The second pier used a steam crane that could be withdrawn at the end of the working day. Blocks were lifted by a scissor grip by Stothert & Pitt (Figure 12.10) acting by friction into recesses in the blocks.

Hayter's main concern was however to discuss instances in the last year or two of concrete failure by internal expansion giving rise to a white cream-like substance (that he later analysed to be magnesian hydrate). He had found these failures rather puzzling, but found an explanation in an article published in May 1886 in the French Academie des Sciences in Paris by G. Lechartier who had identified the presence of magnesia in failed structures, believed to have been included in the cement. He also found similar problems caused by high proportions of lime carbonate, in some instances as high as 12% of the cement. This did not prevent setting, but absorbed carbonic acid from the atmosphere. Hayter argued that tests for cement had been biased excessively towards strength and not enough attention had been given to chemical composition, and the tests needed. He therefore advocated:

- introduction of a chemical test for cement;
- no carbonate of lime to be permitted;
- magnesia to be less than 1%;
- finer grinding of cement, as used in Germany.

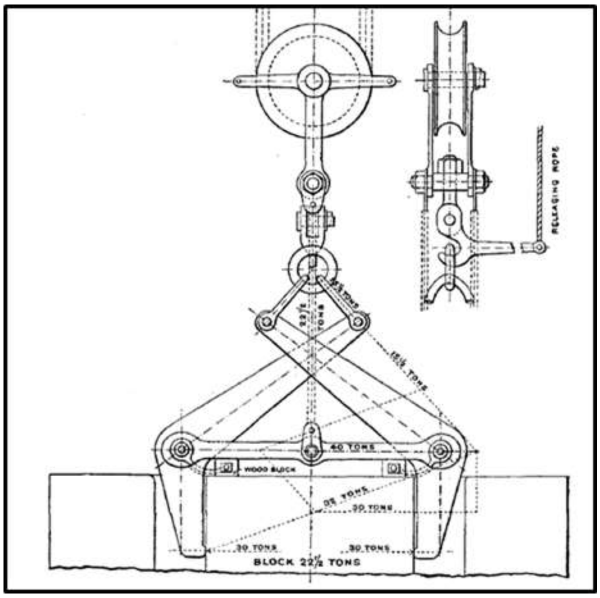


**Figure 12.10 Stothert & Pitt scissor-grip**
*After Hayter in discussion to Carey (1886)*

**J.B. Redman** noted the growth of cement producers along the Gravesend Reach of the Thames over the last 40 years. He noted however that the meeting had so far omitted important work by Cay and by Bernays in using concrete in different ways. He noted that a recent edition of *Engineering*, Vol xlii, "*had described harbour engineering at the present day as almost a disgrace to the profession*", a statement to which he entirely demurred, the writer having "*ignored the great works*" … "*of Smeaton, Telford, the Rennies, Walker and Rendel, down to the days of Sir John Coode*".

**A.K. MacKinnon** described the construction of a seawall in Uruguay using a travelling timber shutter filled with concrete in increments. A local quick-setting cement (not OPC) was combined with lime and sand to fill the voids in granite masonry.

**Colonel Martindale** had much experience in the use of concrete at the Royal Albert Docks, and had observed no abrasion of the dock walls by ships, but rather by loose barges, formed of iron, and often moving repeatedly against the dock walls causing abrasion damage. Even so, it had not yet been necessary to cut out concrete and replace by brick or stone. For new works he would recommend the incorporation of a belt of stone or vitrified brick at barge level to reduce abrasion damage (to the wall).

**J.L. Stothert** observed that piers in the past had generally been constructed from staging, but that this was prone to ship damage, and deeper water had required piles beyond lengths easily found. Engineers at Kurrachee and Ymuiden had devised construction from the completed works using Titan cranes, initially with recantgular motion, only, so no rotation. Recently Messent of Newcastle had however produced a crane that would set 50t blocks at up to 92ft (28m) from the crane centre. He discussed various improvements to concrete mixing, and to cranes, especially the inclusion of sprung wheels to reduce damage, to wheels and rails.

**WJ Chalk** contended that the including of the finer fractions from crushing, dust, harmed the concrete by diluting the cement. He advised removing this dust by e.g. "*a blast of air*". He also noted that including sugar in concrete led to its disintegration – but the reasons for adding sugar entirely escape this reviewer!

**Sir James Douglass** had visited the breakwater construction at Columbo, but did not agree that seas at Columbo were the heaviest known, although he acknowledged the very heavy ocean swell. He felt that, with increasing confidence in the abilities of the Titan crane, even larger blocks might be used, thus reducing the number of joints.

**J.B. MacKenzie** entered a plea for better precision in describing mix design, particularly in separating the sand and gravel fractions as he contended that the concrete strength was adversely affected by higher proportions of sand. He discussed various cases of including the sand with the gravel, or separately. He also reported briefly some experiments on the strength of concrete beams hardening in air, or under water, the former achieving 4x the strength in bending of the latter.

He was also concerned that English engineers were still relying on neat-cement tests for cement quality / strength rather than the more superior mortar tests. He separately noted that the influence of 'dust' on strength depended markedly on the source of such fines. He had found that fines from crushed basalt might be retained as sand without injury.

**A.A. Langley** had constructed quay walls at Lowestoft requiring end-on header blocks every two stretchers to increase strength in the presence of weak sand foundations.

**W.R. Kinipple** discussed the costs of dredging at Newhaven, but primarily the use of cement grout to stop leakages. He had success using a paste of neat cement, rather than "*a thin grout*" which might well separate.

On Scottish fishing harbours, he had used wide breakwaters to also serve as quays and work areas, including net storage in alcoves in the parapet. The hearting between the walls was weakly cemented, "*almost a mass of dry stones with just sufficient cement to bind them together*". At Girvan, the breakwater did not require to act as a quay, so was only 1m wide at the crest. For small harbour works or extensions, large plant could not be afforded, so large blocks could not be used.

In replying to the discussion, **J. Kyle** noted that the heaviest sea-stroke at Columbo occurred not at the head in the deepest water, but part-way out in depths of 5.5 – 7.3m only at 90 – 400m from the root.[126] On blockwork vs bag-work he favoured blocks as giving greater certainty on soundness. He also agreed on the need to exclude 'dust' which would otherwise destroy or reduce "*the cohesive power of the concrete*".

In responding to the meeting, **Carey** clarified several confusions that had arisen in the discussions, and discussed the shape of the sack-blocks in situ. He felt (with hindsight) that they could have been made longer, obviating any tendency for blocks to overhang at the ends. To try to counteract a tendency for the large bags to form a hummock in the centre, the ends of the bags were filled more than in the centre.

In discussing the costs of dredging, specifically their depreciation, he explained the disparities as based on the costs of purchasing the hopper-dredger new, versus the stationary dredger which was an old wooden vessel, also used for pile-driving.

In correspondence, **J. Barron** noted that repairs to the South Pier at Wick had failed. A method previously used successfully at Buckie was adopted at Wick in 5m depth at LW using timber frames lined with jute bagging. Concrete was discharged into these forms from iron skips, casting up to 700t per day.

[126] Sadly, Kyle does not give us any bed slope or wave period to allow shoaling / breaking checks to be made!

## 12.9 Ardrossan extension by Robertson (1895)

In 1806, Telford proposed an outer harbour sheltered by a breakwater; a tidal harbour for 50-60 (small) vessels; and a dock to hold 30 large vessels (300-600t apiece). Progress was slow and the dock opened in 1845. By 1864, vessels were larger and a new dock and tidal basin were required. The harbour was transferred from the Earl of Eglinton in 1886. Site investigation included 230 boreholes. Even so, one of the coffer dams failed on an exceptionally high tide in November 1887 flooding an area of 13 acres (52,000m$^2$). A further cofferdam was used to extend the pier through much of 1888, but much was destroyed in a storm.

Cement was imported from London. Robertson then describes the concrete mixes (1 cement, 1.4 gravel, 2 crushed stones, and 1.2 sand), and method of mixing in McKinnel mixers.

For the breakwater, concrete in 40 or 50 t loads was deposited into jute bags in wooden boxes 8.5m x 1.8m x 1.2m on a steam lighter. These were then lifted by overhanging sheer-legs on a barge to be lowered into position when doors in the bottom of the box were opened. Three bags were placed in October 1887, then re-started in July 1888. In 1889, work restarted in March and continued to October. In 1890, bags were placed from April to December. The average was 4 bags per day, the maximum was more than 8. The total used was 886 bags at an average of 34t per bag. The upper part was formed by rubble concrete in timber falsework (frames lined by timber boarding). A 3t steam crane was used for handling.

## 12.10 Repairs at Alderney by Townshend (1898)

Townshend, who had worked on repairs at Alderney 1889-1890, refers to Vernon-Harcourt's (1873) paper for the main details of the breakwater. He notes the use of granite blocks for the lowest five courses of blocks, and that they can be exposed by the lowering of the foreshore. In repairs in 1898, 12t concrete blocks (1 part Portland cement to 8 parts sand and rubble) faced by 'hard spalls' were substituted for granite blocks. The blocks were cast with the battered front face, and with indents for joggle joints at sides and above. A balancing beam allowed the block to be slid down the face of the wall to its intended 'hole', being held in the correct orientation at the extremity of the counterbalancing beam. Iron rails or bolts set into indents in the new block were filled by Portland cement mortar to connect the new block to previous work. For the foreshore feeding, Townshend notes the limit of 12t imposed by the rail wagons used to convey rock from Mannez quarry.

Repairs in 1889 used a 12t crane, a 5t long-jib crane, a six-coupled locomotive with trolleys. The foreshore was routinely 'fed' using twelve train trucks carrying about 60t of large stones, emptied down the front face in about 20 minutes.

## 12.11 Folkestone by Ker (1907)

Ker's main focus here is on works begun in 1897 and finishing in 1905. He starts by noting the "*vast accumulations of shingle*" caused by the harbour works at Folkestone and that initial works here were authorised in an Act of 1807 for the protection of vessels using the port. He notes that the early works "*consisted of a pier composed of open rock-work, laid as sloping blocks … form the wall of the existing inner harbour*". Further works continued the attempts to keep the harbour entrance open from the ingress of shingle, with the breakwater extended seaward following a further Act in 1818, but again it was outflanked. The accumulation of land to the port, some 26 acres in the 100 years to the time of the paper, had directly provided space for many of the harbour facilities. He comments that the accretion had abated by about 1870, probably due to the construction of groynes further west reducing the arriving drift rate.

By 1897 "*the old pier*" was in some distress. It was formed by a rough mound some 165m long and 9.1m wide formed by loose rock of about 2t. The top of the mound reached about ½ tide, founded at -2mLWOST at the inshore end and -6.1mLWOST at its seaward end. (Spring tide range at Folkestone was reported as ~6m.) Over the mound, timber staging was used to place the rubble, and was then used for traffic on the cross-channel ferry service.

Around 1870, the pier had been widened using sloping concrete blocks, outside of which were driven double Memel timber piles (a Baltic timber, providing long lengths, but with an occasional tendency to rot in the centre). The piles were taken to 4m above high tide, so to +10mLWOST. The tops of the concrete blocks were however only taken to 1m above high water, so to +7mLWOST. The top of the concrete blocks was finished off by bricks on edge paving the uneven ends. In service however the quality of the concrete blocks proved to be "*deplorably bad … rapidly crumbled away …*". Large voids developed which required filling by concrete bagwork before the new works could be completed. These were essentially encasing the old pier by new walls of 6:1 concrete blocks of 16t keyed together by circular joggles 0.25m diameter and 0.61 length.

Ker turns to discussing, at considerable length, the handling of the cement, received by rail in 100t loads. It was stored in bins for at least a month, being turned at least 5 times (and sometimes up to 10 times) in that period to aerate the cement. A boiling test was used to test the integrity of cement briquettes at 7 days, once they had been used to measure their tensile strength. The failed briquette was immersed in boiling water for 3 hours to check for swelling or softening. The cement from briquettes that failed was then rejected. The test was repeated after further storage and aerating, generally leading to improvement in the cement, thus passing the 7-day boiling test. Ker also debates (but without useful conclusion) whether it was wise to add small quantities of gypsum to fine-ground cement to slow its setting. He notes that some fine-ground cements tended to set whilst being worked.

Concrete for the blocks was mixed in two steam-powered Messent 1yd$^3$ (0.76m$^3$) mixers turning our 60 loads each per day. Blocks were lifted at 7 days, and stacked to mature for another 4 weeks.

Construction of new works used timber staging formed by 0.46m square Oregon pine piles often in clusters of four. These carried four travelling cranes, the outer carrying up to 30t, the other three carrying up to 20t. The diving bell (3.9m x 3.0 x 2m) worked by the outer crane weighed 26t out of water including cast iron ballast. Four men worked in each bell, excavating down to the greensand foundation. Helmet divers then supervised placement of the foundation blocks. A 1:24 scale placement model was used to clarify the disposition of blockwork – "*the divers obtained great benefit by seeing in miniature …*". In situ concrete used to fill between the walls used 4:1 mix, deposited in open-topped skips with tripping doors. This set hard except where levelled by divers where it lost cement content during that working.

## Discussions following Ker on Folkestone

**Dyce Cay** described a 100m long steamboat pier that he had constructed at Bressay, Lerwick in the Shetland Islands. At its head the sea bed was at -8.8mLW. The foundation, and most of the wall, were formed by 9t concrete-filled bags laid in stretcher bond for the two lowest courses, then as headers. Each bag was filled in the skip, then deposited in place by opening the bottom door. The walls on each side tapered upwards, the sections above LW being cast insitu. Between the two walls was filled by rubble hearting, and the top of the pier was finished by a concrete deck. Whilst sand and gravel were won locally, the cement had been shipped to Shetland from the Thames. Construction plant were far simpler at Lerwick where they had used a Priestman grab and a steam crane mounted on a single barge from which the bags were laid.

**AC Hurtzig** preferred a vertical face to one battered, but did not give any clear reason for that preference. He commented further on the practicality of aerating cement, and the concomitant reduction in density. He acknowledged that aerated cement might be of better quality, but there would be less of it (by mass)! He also referred to the Messent mixer as 'old-fashioned'.

**CS Meik** asked whether the cement was made by the '*new rotary process*' or the old process. (The author answered by the old process.) He noted that rotary cement was notably fast-setting, and that coal clinker slowed setting of cement from the old 'kiln' process. He discussed further strength tests on samples using either 'kiln' or 'rotary' cements, the latter giving greater strength initially, but reducing in time; whilst the former started at a lower strength, but subsequently gained strength.

**James C Inglis** noted the importance of a batter to a quay wall when used to accommodate (vertically-sided) steamships.

**Sir John Wolfe Barry** noted the competition for ferry trade between Folkestone and Dover, and commented on the easterly drift of shingle which the works at Folkestone interrupted.

**Bertram Blount** noted particularly that the discussion on 'turning' cements dated from 1897 and believed that cement manufacture had since changed to obviate the need. The 'boiling' test for briquettes had been regarded as "*too drastic a requirement*", but was now accepted by many producers. He agreed that the rapid-setting nature of fine-ground cement could constitute a problem, and continued the debate on the wisdom, or otherwise, of adding gypsum to slow the setting. He cautioned that tests on the strength of neat cement (as opposed to on mortar or concrete) with strength decreasing in time did not apply to mortar or concrete where his tests always showed an increase in time.

**AE Carey** discussed movement of shingle along the south coast of England. He noted that "*the practice of removing shingle for building purposes, which had been universally condemned, still went on unchecked*". On costs, he cited the Newhaven breakwater constructed at £60 per foot length, whereas Hasting breakwater had cost £63.3 per foot length. He then made some (possibly rather contentious) comments on cement: "*about 40% of most cement manufactured 20 years ago possessed no cementitious properties*" …"*The fact was that, using cement ground to the standard fineness of today, 8-to-1 concrete was now equal to 5-to-1 concrete of 20 years ago*", i.e. 1887. He asserted that the older cement set more quickly, but quickest when neat, taking 2-5 times longer when in 3-1 mortar or 8-1. He then discussed the addition of gypsum (intentionally or by accident through the clay fraction in the cement). He had found that cement briquettes with a high proportion of gypsum crystals disintegrated in 4-5 weeks.

**HKG Bamber** observed that there had been "*great advances*" in the manufacture of cement, and it no longer needed storage and turning before it could be used. He noted that the British Standard was "*brought into existence during the last 2 years*". Other contributors also discussed the effect of gypsum in cement, and movement of shingle.

**The Author** responded to comments, starting by (angrily?) dismissing the relevance of Mr Cay's pier at Lerwick, exposed to "*waves no more than 3 feet*", to conditions at Folkestone with waves "*measuring 13 feet*". He thought that the concrete bags used at Lerwick would have been impossible at Folkestone where "*the seas would play havoc with the side walls and the rubble hearting*". He answered several questions posed by other discussers, and he hoped that discussions on cement quality, the need (or not) for turning, and inclusions of gypsum, would be illuminated in the future by results of longer-term testing.

## 12.12 Dover by Wilson (1919).

The ancient ‘Pharos’ (Roman lighthouse) at Dover dated from AD46 is claimed to be the ‘*oldest building in England’*. 1840 Royal Commission reported in favour of establishing a deep water harbour in Dover Bay, enclosing 450 acres at LWOST. Estimated cost was £2,000,000. Further Royal Commission in 1844 considered whether it was desirable to establish a 'harbour of refuge' in the channel; and then, which site would deliver, in order of precedence:

a) Ease of access for vessels 'requiring shelter from stress of weather';
b) For armed vessels in event of hostilities, both offensive and defensive;
c) It should *'possess facilities for ensuring its defence'* against attack.

Whilst this harbour was in theory to be for refuge for civilian vessels, the military purposes are clear from the start. This second Commission accepted the proposed site and general plan layout of the new outer harbour.

A third Commission in 1845 considered plans by eight leading engineers, and again accepted the proposed site. The issue of siltation was again considered, clearly of significant concern, and this commission commented rather testily: '*... if liability to silt were deemed an objection, it would be idle to attempt such works on any part of our coasts*'. A construction contract was let in October 1847 for 800ft (244m) of Admiralty Pier. Subsequent contracts in 1854 and 1857 covered further 1000ft (305m), so that in 1871 the work on Admiralty Pier was essentially complete to a length of 2100ft (640m) from the shore. Wilson gives no details of the contractor, or designer.

Admiralty Pier was formed by 7-8ton concrete blocks with stone facings on the outer faces. The main wall was '*surmounted by a high parapet, overhanging considerably to the seaward*'. The paper shows a section of this parapet wall consisting essentially of a single column of blocks, about 5ft (1.5m) thick, with a relatively slight recurve to modern eyes. But about 1000ft (305m) of this parapet at the outer end was swept away down to quay level on 1 January 1877. Wilson ascribes the blame to the curved overhang, although the slender nature of the up-stand wall, and absence of any tensile reinforcement against bending stress, must surely have contributed substantially. The damaged section was rebuilt with a significantly thicker (about 11ft, 3.4m) vertical face, and "*has proved perfectly satisfactory*" (up to 1919).

This single pier did not however give adequate shelter from easterlies, and a contract was let by DHB in 1892 to Sir John Jackson to construct the Prince of Wales Pier to some 1650ft (503m) supervised by Coode, Son & Matthews.

In late 1895, Coode, Son & Matthews were requested by the Admiralty to prepare surveys and drawings to facilitate the expansion to the full Admiralty Harbour. Surveys were complete by autumn 1896, both of seabed levels, and composition, and measurements of tidal currents using floats (at surface and at a depth of 15ft, 4.6m). It was noted that the flood current only starts about 2 hours before HW and continues to 3.5 hours after HW. Conversely, the ebb current runs for 3.5 hours before LW, and 3.5 hours after LW. The spring tide range is given as 18' 9" (5.7m) and the neap tide 15" (4.6m). The new works were:

a) Extension of Admiralty Pier by a further 2000ft (610m);
b) A detached breakwater, the South Breakwater, of 4212ft (1284m);
c) The Eastern Arm of 3320ft (1012m).

This revised design (in comparison with the 1844 plan) altered the length and overlap of the Admiralty Pier extension, and the position and width of the Eastern entrance, with the aims of improving accessibility to vessels, and reducing siltation. The design was approved by the Admiralty, and a contract let for construction to S Pearson & Son in November 1897.

These walls were essentially formed by concrete blocks (1 part cement to 6 parts sand / shingle). Blocks were 7.5ft (2.3m) wide and 6ft (1.8m) high, depth varying from 8 to 14.25ft (2.4-4.3m) to accommodate the 12:1 batter, and ensure adequate bonding. Jointing was further strengthened by half-height joggle joints, filled by 4:1 concrete rammed into canvas bags. Around the outer ends, further tensile strength was provided by bull-headed rails turned down at the ends over 12" (0.30m), and let into chased channels / holes, then filled by 2:1 cement mortar.

For the foundation layers, some block depths were changed to 4ft (1.2m) from 6ft (1.8m), allowing the foundation level to be varied in 2ft (0.6m) increments. Underwater blocks were set by divers, placed tightly without mortar. Above the low water course (a band 6ft (1.8m) high centred on LWOST. Next four courses up were grouted by 2:1 Portland cement mortar. The Eastern Pier and Admiralty Pier Extension carried parapet walls, but such additional overtopping protection was not needed on (most of) the South Breakwater as mooring against its inside face was not envisaged. Mass concrete and granite pavers completed the crest. The parapet wall on the Admiralty Pier Extension reached +24.5ft (+7.5m) HWOST, so +43.5ft (+13.3m) LWOST. Given a harbour cope level of +10ft (+3.05m) HWOST, this gives a parapet wall height of 14.5ft (4.4m).

The reclamation at the root of the East Breakwater was used for block production, preparation of in-situ concrete, and other construction activities. Temporary works gave shelter that would later be provided by the main breakwaters. Care was taken to fill plan re-entrant corners with pitch-stone slopes to avoid local concentrations of load / overtopping.

The Eastern Breakwater, also termed the East Arm, projects south for 2942ft (897m). The section is essentially similar to the Admiralty Pier Extension, although the parapet wall was lower (height not given!!). The harbour cope was set at +10ft (3.05m) HWOST (so approximately +29ft (8.8m) LWOST). The foundation blocks of the East Arm wall were laid direct on the chalk inshore, or the chalk marl / flint matrix further seaward, down to -53ft (-16.2m) LWOST.

The East Arm was intended to provide berthing, so the harbour face was made vertical, timber fenders were provided, and an L-shaped head (50ft, 15.2m) provided some shelter to wave action along the inner face. One assumes that the harbour cope was again set at +10m (+3.05m) HWOST.

The South Breakwater, occasionally termed the Island Breakwater, runs 4212ft (1284m) generally parallel to the shoreline. Interestingly, block placement for this wall was commenced short of the eastern end, allowing a later adjustment of the width of the eastern entrance on the basis of wave penetration and flow experience during the construction period. A curved section connected the eastern end to the main run of wall using curved (radial) blocks to ensure the preservation of block tightness. No parapet wall was used along the main section of the South Breakwater, simply being added at the ends to provide shelter to buildings close to the roundheads.

Aggregate (termed ballast in this paper) was obtained from Sandwich via the Deal & Ramsgate Railway, with some supplied from Dungeness, and later from Rye.

Cement (mostly from the Wouldham Company[1]) delivered by barge in 160t loads was derived from '*ordinary- and rotary-kiln*' production. Wilson notes that the rotary-kiln cement was '*usually far quicker setting*', so the two types were mixed. Concrete was mixed in two electrically powered 'Messent' mixers of 1 $yd^3$ capacity (perhaps the first use of electric power for such an operation. Output averaged 100$yd^3$ per mixer per day.

Blocks were lifted after 7 days, and then stored for at least a further 3 weeks. Two lifting holes ran through each block for the T-headed lewis bars used in lifting / release. Blocks within the storage yard were moved by two 42t travelling Goliath cranes, carried from there on stripped down old (steam) locomotives.

Granite-faced blocks used a face of granite cast into the rest of the block. Granite blocks were mainly supplied from a Pearson-owned quarry at Gunnislake in Cornwall, supplemented by a supply from Sweden, requiring special permission from the Admiralty – although why is not revealed.

Pearson eschewed the use of Titan block placing cranes that would run directly on top of the constructed works, in favour of temporary staging above and beyond the works, supporting a number of Goliath and other (steam powered) travelling cranes weighing (unloaded) between 220 and 400t, Figure 12.11 and Figure 12.12.

**Figure 12.11 Dover - placing concrete blockwork from travelling cranes**
*Courtesy Dover Harbour Board*

The rail level for these cranes was generally above +27ft (8.2m) HWOST. The (mostly Tasmanian Blue Gum) supporting piles were upwards of 100ft (30m) long, of 18" or 20" (0.46-0.51m) square section. Wilson notes that the Blue Gum timber was heavier than water, but Oregon pine when used required weighting by old iron rails to ensure sinking. The staging piles were re-used as the work progressed along, being extracted by a floating hulk with a jib and powerful winch. After use, piles were spliced to ensure availability of an undamaged head for driving.

**Figure 12.12 Dover - placing concrete blockwork from travelling cranes**
*Courtesy Dover Harbour Board*

The Goliath cranes used out on the staging, and within the blockyard, were essentially similar, being of 40, 42 or 60t capacities. A winch engine travelled on two cross-girders some 100ft (30m) long, themselves travelling along the staging. The winch engine controlled the transverse position as well as block lifting / lowering.

Ahead of block placing, the seabed was prepared for the foundation by excavating 4-5ft of surface material, most by a 'Hone grab'. The chalk or chalk / flint matrix was loosened where needed by a cast-iron breaker fitted with protruding chisels, and dropped from the leading (60t) Goliath. The final 0.3m of excavation was removed by four men using picks and shovels within a large diving bell (17.5ftL x 10ftW x 6.5ftH) weighing 35t (in air). These excavated a strip about 15ft (4.6m) wide across the running face, sufficient to place two rows of blocks. The bell was passed over each strip twice to give a coarse levelling, "*within a few inches*", and then a second pass for final levelling.

This working, under compressed air supplied through flexible pipes / hosing from the staging above, continued day and night in 3 hour shifts. The safety culture of the period might be gauged by "... *it is satisfactory to note* *that although there were several cases of illness amongst the bell-men, it was no means excessive when the large amount of time occupied in this work is taken into consideration.*"

Block-setting was supervised by two helmet-divers, blocks being placed hard against their neighbours. Significant effort was devoted to checking and regularising these courses to ensure an even base for the subsequent blocks. Bag joggles were placed by the divers, or from within the bell returned to deal with several blocks, and to regularise any unevenness in the completed surface. Helmet-diver working was limited to flow velocities below 1 knot, restricting these operations to about 4-5 hours each tide, during which 6 blocks were placed per hour at best.

Further trimming or filling at the base of the 'Low-water course' compensated for any level errors in the lower layers. Blocks above were set by masons during the 2-3 hours of low water on spring tides. All the upper courses were set / bedded in 2:1 Portland cement mortar so significant effort ensured that all lower joints were caulked by sacking / rope, pointed in neat (quick-setting) cement, to avoid any loss of jointing / bedding mortar downward.

Toe protection blocks were laid along the seaward face using essentially similar procedures with a smaller diving bell operated from the small luffing-jib crane running along the completed surface of the wall. As these protection aprons were completed, so the parapet walls were added above. A capping layer of in-situ concrete with granite paving completed the deck, allowing for rails, gas / electric / telephone cables and water pipes.

Wilson notes that on an average length of staging of 1300ft, six significant machines were:

One traverser with 20t derrick crane;

Four 40t Goliaths; and

One 60t Goliaths

During construction, various shipping accidents caused damage to the operation. The warning lightship at Admiralty Pier was run down at least four times, in two of which it was sunk! The South Breakwater was struck twice, once removing three bays of staging, a traverser and the outermost Goliath. In November 1907, a liner (name not given by Wilson) struck the recently completed western head, damaging 80ft length across the full width, moving around 10,000 tons of blocks in a single mass. This section was then re-constructed, requiring re-erection of the outer bays of staging.

Wilson concludes the paper with a full list of acknowledgements, and notes that the total cost was £3,500,000, and that the extended harbour was formally opened in November 1909 by the then Prince of Wales.

## Discussion to the paper

The ICE President (Sir John Aspinall) noted a proposal by the 2nd ICE president, James Walker, to form 300ft (91m) sections of breakwater in caissons and float them around from Portland. That proposal was not adopted.

**Sir William Matthews** clarified some aspects of the history, particularly of the sequences of expansion of the 'commercial harbour' by DHB, and of the Naval Harbour by the Admiralty. He noted that construction of the latter had been under consideration since 1845, but that by 1890 DHB could no longer delay improvements required by improved / larger boats on the cross-channel services. DHB therefore requested Sir John Coode to provide a design for an extension to Admiralty Pier, and an eastern pier, later the Prince of Wales Pier, and berths for four large mail-boats. Having constructed about half-way along these works, about 2-3 years later, "*the Admiralty decided to proceed*". [127] Matthews discussed the complications of the changes needed, further discussed the potential for siltation, and of wave action within the harbour (both diffracted and locally-generated).

[127] One can almost hear the frustrated tone some 110 years or so later!

**Admiral Sir Reginald Bacon** reviewed the military need for the harbour. He noted that "*relations of Great Britain with France at the end of the last century were traditionally bad … and it was necessary to have safe harbours on the south coast to cover the north coast of France.*" He noted that in 1914 about 25 vessels were based in Dover, but by 1917 nearly 400 were based there. During the war a mine-field was laid in (this area of) the Channel every 4 days. He however also noted that westerly swells still significantly penetrated the harbour such that moored destroyers might roll 20-25° each way.[128] He noted the later sinking of a vessel at the western end of the South Breakwater to disrupt swell penetration, believing this modification to be effective. He concluded that "*the harbour was a good one, generally speaking; it was not a comfortable one, and everybody hated it, … he always told captains that if they did not like the inside they should go outside … nobody, however, ever went out…*"

**Sir Ernest Moir** had started on the design of the plant and machinery, but had in 1900 taken over executive responsibility from Lord Cowdray, being there from the setting of the first block to the last, indeed of the last block twice as it had to be reset after the ship moved 10,000t. On issues of wave penetration, Sir Ernest noted that the Navy often put their destroyers in 14ft (4.3m) of water when "*it was common knowledge that the vertical motion of waves in deep water was transferred into horizontal motion in shallow water, and of course the destroyers rolled.* [129]

During the war, there had been considerable concern on the possibilities of submarine attack through the two entrances, and it had been suggested to block one entrance permanently, with submarine nets across the other. In the event the western entrance was modified by sinking two ships approximately parallel within the entrance, across which submarine nets could be operated. Permanently steamed winches allowed them to be lowered / raised in 40 seconds at little notice.

On construction progress, he noted the need for night and day working, with the divers doing all their work in darkness, averaging six blocks per man per hour.

Sir Ernest concluded by praising the designers, especially the quality of their drawings, particularly those for "*the quarry point of view, so that the mason did not have to cut all his stone to one size, and did not find that larger stone was specified than any bed in the quarry would yield.*"

**E. Cruttwell** asked about the mechanism by which excavation material was disposed from the diving bell. Claiming experience in connection with harbours for commercial use, he commented on the use of granite facing, speculating that this was rather expensive and that concrete faces might have sufficed. On wave penetration, he wished that Admiralty Pier might be further extended, and felt it would be better if the Eastern Entrance were to be blocked up. [130]

**A.T. Walmisley** discussed the interaction of the commercial harbour developments with the Admiralty Harbour. He noted that aspects of the Admiralty Harbour construction had been copied in the reclamation scheme which followed it. The base width of the Reclamation Wall was equal to half its height as it had to stand alone before the chalk filling between it and the old pier. He praised the staging used in the main construction, noting how useful the cross-girders were in setting out and measuring work below. In discussing the use of mass concrete foundations, he noted those of Seaham Harbour piers formed in mass concrete 33ft (10.1m) or 40ft (12.2m) wide, but he felt it better to carry the blockwork downwards as at Dover wherever circumstances permitted.

**C.H. Colson** discussed flows, sedimentation and dredging. He refers to 'a locomotive stage used perhaps by Hill (of Hull) from which holes could be drilled much cheaper than from floating barges. He noted construction

[128] Sir Ernest Moir countered this later by explaining that it arose from the way / place that the Navy chose to moor their vessels.
[129] Admiral Bacon admitted that the moorings had now been altered.
[130] One suspects that the development of ferry operations at Dover Eastern Docks might have been somewhat inhibited!]

of the submarine harbour using 6t blocks, but that they cost as much each to set as the 40t blocks on the 'big breakwater'.

**L.H. Saville** noted that "*nothing had had to be provided in the (Admiralty budget) estimates for repairs*", which he contrasted with 'other breakwaters … in different parts of the world'. He noted that the Admiralty proposed to retain the two blocking ships at the western entrance, monitoring their effects, and possibly replacing them by some permanent structure.

**Wilson** responded to some of the points made by discussers. He disagreed with Cruttwell that there would be any benefit in closing either entrance, noting Colson's comments on the beneficial flows. He did not agree that the use of granite facings was extravagant, supported by Saville's comments on the lack of repairs needed. He confirmed that excavation material was generally lifted with the diving bell, but was occasionally used to back-fill the edge of the excavation against the foundation blocks.

## Correspondence to the paper

**Asa Binns** noted the 55 years during which hardly anything happened, the design completed in 2 years and construction in 12 years. He regarded the increase of block size from 7-8t on Admiralty Pier to 40t on these breakwaters as "*of great importance for the permanence of the works and reduction of maintenance costs*." He noted the boldness of using staging and Goliath cranes rather than building over-end with Titan cranes. He wondered whether any advances (in payments?) had been forthcoming to cover the (several hundred thousand pounds) value of the plant.

He noted the implicit uncertainty in the proportion of water in the concrete (18-20 gallons per 1 $yd^3$). He had noted an average of 18 gallons per 1 $yd^3$, and particularly commented that "*an excess of water was highly prejudicial to the quality of the concrete*". He then referred to a (probably on-going) debate on British Standard classifications of setting rates of cements.

**W. Dyce Cay** discussed the use of two entrances which he also felt to be disadvantageous. On the bag joggles, he preferred full height versions as he had used for 25t bocks on the roundhead of the South Breakwater at Aberdeen. He contended that the 2:1 mortar retaining the iron cramps "*would not set hard*", although why not he does not explain.

**Sir Whately Eliot** especially noted the advantages arising from the use of the substantial staging. On the Tyne piers, the staging and travellers were lighter, and the diving bells had been significantly smaller. He endorsed the use of the protecting scour apron as "*he had seen much damage caused by the neglect of that precaution, for the downward scour from the impact of a wave on a vertical wall was most destructive*".

**R.F. Hindmarsh** also alluded to works on the Tyne, where he noted that 6ft deep (40t) blocks had been disturbed 20-25ft (6.1-7.6m) below LW. He was intrigued as to whether the 3.5ft blocks at Dover had been disturbed. He continued the discussion on the need for granite facings and whether the plain concrete faces on the lee faces had suffered damage. He also noted repeated damage experienced on the North Tyne Pier to the promenade surfacing under heavy overtopping. The Tyne Commissioners had abandoned use of asphalt, replacing it by concrete paving.

**J.C. Larminie** discussed flows through the entrances, particularly their effects on vessels navigating across the tidal stream.

**James Mitchell** also considered the need for two entrances, and difficulties of navigating across a strong tidal stream. He noted the advantage of the substantial staging, and machines working from it, particularly when simultaneous working on the foundations and setting blocks was needed.

**Sir Francis Spring** drew contrasts with breakwater construction at Madras which had used a 15ft (4.6m) thick foundation mound, and slice blockwork, occasioned by the lack of hard foundation stratum at Madras. He continued the discussion on the use of ½ height joggle bags, himself preferring them to be full height to reduce the risks from partly filled joggle joints. He bemoaned the absence of any cost information in the paper.

**The Author** responded thanking the various correspondents. No specific advance had been made for plant in general, although some allowance was made for the import of timber. He countered the arguments of those who preferred a single entrance, suggesting that the through circulation had reduced siltation. He failed to see any advantage in Mr Cay's suggestion of a part height weir across the eastern entrance. He also disagreed with Mr Cay on his objections to part-height joggles, noting the monolithic nature of the blockwork moved by the ship collision, and the successful use at Peterhead. The use of rubble foundation would have given little or no advantage as the depth needed to ensure stability of stone in the mound would have been close to the full depth at Dover.

[1] **Note on Wouldham Cement Company (from: http://www.cementkilns.co.uk/)**

The plant had been acquired in 1898 by Pearson's, who wanted their own source of cement for several large projects, principal among which was the upgrading of Dover Harbour. It was taken over by Blue Circle in 1911.

•1874-1880 Lion Cement Works Co.

•1880-1898 A. D. Robertson and Sons Ltd

•1898-1900 S. Pearson & Son Ltd

•1900-1911 Wouldham Cement Co. (1900) Ltd

•1911-1976 BPCM (Blue Circle)

The Wouldham Court Cement Works opened in 1847, and was based on the riverside in the centre of Wouldham village. In 1855, the first owners of the cement works, Thomas Freen and Co, became bankrupt. In 1856, the Wouldham Patent Portland Cement Co (who had taken over the works) leased the land necessary to build a tramway to connect the works with the chalk quarry, which was 1,380ft away. The main commercial cement works owned and operated by Pearson (1898-1900) were not at Wouldham but at South Stifford, Essex. The Wouldham Cement Company (1900), Limited, was formed as a Joint Venture between Pearson's and John Bazley White's (and subsequently APCM) whereby the plant was allowed to participate in the purchase of rotary kilns. The company did not join APCM, but was one of those with a formal "working relationship" with the combine.

## 12.13 Portpatrick by Cunningham (1977)

Initially called Portree, separated from Portslogan in 1628, then named Port Montgomery, (probably) being named Portpatrick by 1648. Informal and irregular ferry services between Portpatrick and Donaghadee (34km) were running by early 1600s. Weekly postal service established in 1662 by Robert Mein (Edinburgh Post Master), running twice a week from 1677. The Post Office then built (or more likely commissioned?) construction of a pier on the south side of the harbour in 1774, designed by John Smeaton. A lighthouse followed in 1790, at which point the mail service became daily using the Post Office's own vessels, and supplied by daily coach service from Carlisle and Edinburgh.

Telford visited to consider potential improvements to the harbour in 1802. His report was studied by Government who held an enquiry which recommended detailed surveys, started in 1814 by John Rennie and completed in 1818. An extended harbour with '*two great piers*' was recommended and was consented and started in 1820. This subsumed Smeaton's South Harbour by extended North and South breakwaters forming south and north basins. The extended South Pier and new lighthouse was complete in 1836, but a storm in 1839 undermined the outer end of the beakwater, endangering the lighthouse. Effort was diverted from constructing the new North Pier to repair the South Pier, halting further expansion. Meanwhile the ferry route was operated by steam packets from 1825 using paddle steamers *Dasher* and *Arrow*, later *Fury* and *Spitfire*. Operation of these transferred from the Post Office to the Admiralty in 1837, but the mail contract transferred to G & J Burns in 1848 who operated the service from Glasgow to Belfast, withdrawing the mail steamers. The ferry service reverted to local sailing vessels.

Lord Stair resigned as chairman of the harbour commissioners in 1847 (perhaps in protest at the imminent loss of the mail contract?), and the commissioners were disbanded in 1849, leading to (?) the adoption of the harbour by the Admiralty in 1850. The Portpatrick Railway Company was formed in 1857, opening the line between Stranraer and Castle Douglas (junction with the Dumfries to Carlisle line) in 1861. The mail service was reinstated and construction of a new dock for mail steamers was approved in 1859. By August 1862 the line into Portpatrick was opened, although construction of the new dock by blasting the whinstone rock was not completed until 1865.

By October 1862, the postal traffic transferred to the Stranraer to Larne route, again cutting out Portpatrick. Despite this setback, the Donaghadee & Portpatrick Steam Packet Company was formed in Belfast, making two trips a day using the *Dolphin* from July 1868, then one a day from September, but ending in October 1868. Various attempts were made to re-establish the route were made in 1871, 1873 and 1891. Meantime in 1868, the Government compensated the Portpatrick Railway (£20,000) for the withdrawal of support for the ferry. The lighthouse was dismantled in 1871, to be re-erected at Colombo (Sri Lanka), and in 1873 the harbour acts were repealed leading to abandonment of any external interests in the harbour. Use continued by local vessels, and a lifeboat station was established in 1877.

See also Paxton & Shipway (2007) summarised in the Geographic section, Chapter 12.

## 12.14 St Catherine's, Jersey by Davies (1983)

This book by a Jerseyman is primarily on the development of the harbour (and breakwaters) of St Catherine's on the east coast of Jersey, but includes bibliography of various personae dramatis and discussion on the UK government policies of 'harbours of refuge' with particular reference to those in the Channel Islands. Davies does not disguise his view of the St Catherine's harbour as a major error and waste of money, and seeks to pin blame where he can. The main activities took place in 1840 to 1850, a period of '*perfect engineering panic*' (Richard Cobden MP cited by Davies).

The UK government was (generally?) happy to support the claim that 'harbours of refuge' were required to shelter British (and other) vessels from storms. In truth, very few if any of those harbours discussed would have been useful for that purpose, Peterhead and perhaps Holyhead excepted. Those around the English Channel (existing or putative) were first and foremost of military significance, particularly those in the Channel Islands which were primarily focussed on countering the perceived threat of the large French naval port of Cherbourg, but also of raiding parties from St Malo. *Sir Thomas Acland MP noted in Parliament that: "It was of great importance to have a station at Alderney for watching Cherbourg"*. It is probable that the initiatives to establish (naval) harbours on both Alderney and Jersey would have been driven by a loyalty to defend those islands against French invasion, and a desire to be able to keep the French navy bottled up in Cherbourg to avoid attacks on British (and allied) vessels plying the English Channel. The Duke of Wellington particularly supported those in the Channel Islands who feared a French invasion of the islands. In the event, this treat faded and the development of harbours at Alderney and (partially at) St Catherine's served little or no useful military purpose.

Alderney – original budget was £600,000, of which £150,000 had been spent during 1847-1851. At which point the Richard Cobden MP suspected '*that the final figure might well approach £1 million*', and was '*sceptical of any other function it might fulfil*'.

Davies cites in particular four personalities of importance to the creation of harbours at Alderney and St Catherine's: (Admiral) Martin White, the surveyor; Admiral Sir Edward Belcher; James Walker, civil engineer, and later President of ICE; and Thomas Jackson of Jackson & Bean the contractor.

Martin White was born at Hayling Island (probably) in 1779, first going to sea in the navy in July 1794, and qualifying as a Lieutenant in December 1800. His first command was (probably) the *Pigmy* which helped blockade Le Havre, St Malo and Granville, during which he '*surveyed at the same time the approaches to each*'. He was promoted to Commander and given command of the *Vulture*, Guard Ship on the Jersey Station, from which '*every opportunity was taken … to obtain correct soundings*'. This activity continued with commands of *Fox* and *Shamrock*. A *Survey of the Channel Island and the Coast of France* was submitted to the Hydrographic Office in 1815, and his *Sailing Directions for the English Channel* published in 1834 became the definitive guide. His surveying abilities were held in high regard, being especially commended by Captain Thomas Hurd, Hydrographer to the Navy; and Admiral Sir George Cockburn, the senior Naval Lord. Martin White retired from active service in October 1846 having surveyed some 60,000 square miles of ocean. He was promoted to Admiral (Retd) in 1862 and died in June 1865. During and after his active service, he was consulted both formally and informally as to the navigability (and perhaps utility) of proposed harbours around the Channel Islands.

With a tidal range of order 12m on spring tides, none of the Jersey harbours were navigable at all stages of tide, generally being half-tide harbours with vessels drying over low water. This could be tolerated for commercial activities, but not for naval use where the necessity for sheltered moorings in deep water was realised in about 1830 – '*but steam altered all previous reasoning and the need for a harbour of defence became obvious locally. It took a little longer to get through to Whitehall, and then for diplomatic reasons it was called a harbour of refuge.*'

In 1840, Sir John de Veulle (Baliff of Jersey) referred to *"the enemy's fleet"* in writing to the Admiralty, doubtless meaning the French Fleet, building pressure in Whitehall for UK government action to strengthen defence for the Channel Islands. The perception of such a threat was substantially increased by French plans (and actions) to strengthen / expand Cherbourg, St Malo and Granville. A seminal document is the States of Jersey Royal Petition of 26 August 1840 which protested the loyalty to the English Monarchy of the Island of Jersey; that the advent of steam power "*totally altered the plans of defence*"; and that the "*gigantic efforts made on the opposite coast gave little solace to the islanders*". In conclusion, "*Her Majesty was asked to arrange for the construction of a harbour without delay*". Within Whitehall, the petition "*went astray in the tortuous corridors … this was not looked upon as a calamity*" – stalling tactics again? In June 1841, the Lords Commissioners of the Admiralty discussed "*whether there ought to be a Pier and Breakwater at Jersey or not …*". Whitehall's response to the Bailiff "*was accidently sent to Sir Edward Gibbs, Lieutenant-Governor instead of the Bailiff*"! Sir Edward was not enamoured of the idea of a harbour of refuge, perhaps because he foresaw the requirement for substantial expenditure by the island to provide local infrastructure and military works solely in support of such a harbour. Worse still if such a harbour was planned for the north coast of Jersey, perhaps Bouley Bay. It is noted that St Catherine's Bay was not mentioned at the time, the favourites being Noirmont Point or Bouley Bay. [As a side discussion, potential cost savings from use of convict labour was inspired by use of prisoners in the construction of the harbour at Portland. Similar issues were (later) debated in the ICE discussion to the paper by Scott (1858) on Blyth.]

The issue of a new harbour on Jersey was then complicated by the involvement driven by the Home Secretary of Sir William Napier, Lieutenant-Governor of Guernsey, who appears to have been inveigled *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, and he was certainly unpopular*". The UK government then set up a Commission to revisit

Sir William's work. One need not speculate much on how that went down with the Lieutenant-Governor. Members of that Commission 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 which included Captain (later Rear-Admiral) Sir William Symonds (also surveyor).

But by 1842, Government was minded to act. There were competing claims for Noirmont Point on the south-west coast or Bouley Bay towards the north-east corner, or none at all. For reasons that a still opaque, and not supported by any of the protagonists, the government opted to construct the new harbour at St Catherine's Bay on the northern part of the east coast of Jersey. Davies notes that the reasons to go against Martin White's advice for Bouley Bay "*have not been easy to trace because Admiralty papers on this delicate subject have been ... 'weeded'*", although this might be a bad dose of conspiracy theory.

Davies notes that the Harbours Commission (Committee?) of 1844, set up by the Lords Commissioners of the Treasury to investigate suitable sites in the English Channel, 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 2 April 1844 and the report being submitted to their Lordships on 7 August that year. [Perhaps much work had been completed in advance of the Act being passed.] The 'omission' is shown by the absence of any reference in the report to the Channel Islands, interest being confined to Portland, Seaford, Dover and Harwich.

It was William Symonds (later Sir) who had written to the Lieutenant-Governor in 1831 assessing the naval activities on the French coastline, and appraising options for harbours around Jersey. In this he favoured Bouley Bay, although this had been at the time countered by Martin White who "*unmistakably showed up the defects*" of the north-east option. In discussing these options, Davies reminds the reader that developments of steamships were in their infancy in 1831, but that over the following years the changes in requirements for harbours were significantly modified to deal with the changing forms of propulsion, particularly the reduced mooring and swinging space required, and the improved ability to depart under adverse wind directions. This was of particular 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.

Discussions on the location of any such harbour on Jersey were now (1845-46) significantly complicated by various proposals for railway lines, generally hugging the south and east coasts, and circumventing or tunnelling through / below a number of natural obstructions and/or castles! To this end, James Walker was retained to report on a railway proposal linking the south and east coasts: presenting "*a very good example of saying absolutely nothing of importance in a manner which nevertheless suggest that one cannot afford to ignore it*". The report was issued in October 1846, and despite it *saying nothing*, within days the Secretary of the Jersey Railway Company concluded that "*the undertaking must be abandoned.*" Intriguingly, there appears to have been an expectation that the same Railway Company might pay Walker's "fanciful account" of above £5000, despite his instructions emanating from the Admiralty. Davies concludes that "*the report was not worth a fraction of 1% of this amount, being quite useless for all practical purposes*". So having ignored the option of a new breakwater at Noirpoint (supported by Martin White), or the less-logical north coast option at Bouley Bay, and having abandoned any circumferential railway to bring troops, supplies or even commercial traffic to the new harbour, the decision appears to have been made to build it in St Catherine's Bay, a site favoured by nobody!

In his Chapter 7, Davies endeavours to finger the guilty, despite "*a thorough cover-up exercise ... carried out in Whitehall.*" His main culprit is Captain (later Admiral Sir) Edward Belcher, aided (perhaps abetted) by Colonels Cardew RE and Colquhoun RA who conducted a survey in 1842, and "*made a joint report*" that recommended St Catherine's Bay. This is not however all the truth as Belcher (clearly a forceful character) "*submitted a personal and entirely separate report to the Admiralty*" which included a number of erroneous statements on possible passages around the east coast. [It is worth noting that a line of rocks / shallows higher than -10mCD stretches fully across from the south-east corner of Jersey at Seymour Tower across ESE to Coutances on the coast of France. This effectively blocks any north – south passage from or to St Malo and Granville, except over high water. This implies that any French attack on Jersey would be much more likely on the south coast, say Noirmont and St Helier, than on the east.] Before construction was started or underway, Belcher took command of the Samarang surveying in the East Indies, so "*was not easily accessible when the awful truth dawned*" of the uselessness of a harbour at St Catherine's.

Reasons to develop both Alderney and St Catherine's at the same time (and apparently in a rush) were probably not all defensive, but may have been drive by offensive planning, which would explain in part the lack of clear documents justifying the decisions to construct. Analysis of the potential of St Catherine's Bay from which to mount an attack on the Cherbourg peninsular were probably driven by Belcher, and may well have surmounted any thoughts as to any defensive role.

James Walker had been instructed by the Admiralty to appraise possible harbours on Jersey, Alderney and Guernsey in July 1844. This was followed up in September 1845 by the Board of Admiralty which instructed him to purchase land on "*Jersey and Alderney to enable the construction of harbours at St Catherine's and Braye respectively*". Purchases started in 1846, including land for quarries, "*offices, workshops, barracks, storehouses and all the other paraphernalia that go towards the making of a naval dockyard*," leading to a holding of 274 acres around St Catherine's Bay at a cost of about £72,200.

At this point Davies diverts to consider Walker, who takes the civil engineer responsibility for the scheme. Walker, (http://www.gracesguide.co.uk/James_Walker) was born in Falkirk in 1781, and articled in 1800 to his uncle, Ralph Walker, who ran an engineering practice in London. He succeeded his uncle and inherited many of Telford's projects / clients in 1834. He specialised more in canals and harbours than railways, and was responsible for the Bishop Rock lighthouse off the Scilly Isles. He was held in high regard by the civil engineering profession and held office as the second President of ICE (following Telford) for eleven years, after which "*he was a not infrequent attendant at the Meetings*".

On Jersey, Walker was responsible for the Victoria and Albert Piers, 1841-1847, continuing on to the St Catherine's project four months later. On which Davies noted the "formidable team" of Walker and Admiral Belcher, awarding Walker "*as much skill in the engineering of man as of matter*", continuing to describe him as "*an able manipulator of men*". Davies is also free with his criticism of Walker in his backing for St Catherine's, particularly on issues of adequate depth, compounded by siltation or deposition "*which brought about its ultimate abandonment*". Whilst Belcher and Walker worked together to promote the St Catherine's scheme, they disagreed on the harbour development at Alderney where Belcher preferred Longy Bay on the south (sheltered) side of the island, writing in 1842 "*I am fully satisfied that no harbour could be constructed on its northern shores … it would be constantly subject to destruction of its base …*".[131]

On Alderney, Davies notes the similarity of the design, although also the significant differences in exposure leading to much greater difficulties in constructing at Alderney, and the far greater levels of scrutiny given by e.g. Vernon-Harcourt (1873, 1885) a previous Resident Engineer there. He notes that the initial design at Alderney was soon seen to be insufficient, quoting W Parkes (Resident Engineer at Alderney before Vernon-Harcourt) who observed "*that the inadequacy of the original designed section to withstand the great force of the sea became apparent*". In taking Vernon-Harcourt's side in the discussion, Davies identifies the difficulties in securing the base of the wall, although he lacks the clarity on how and why that instability was caused[132].

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[131] Perhaps James Walker rued favouring of the northern site at Braye Bay.
[132] As did Vernon-Harcourt.

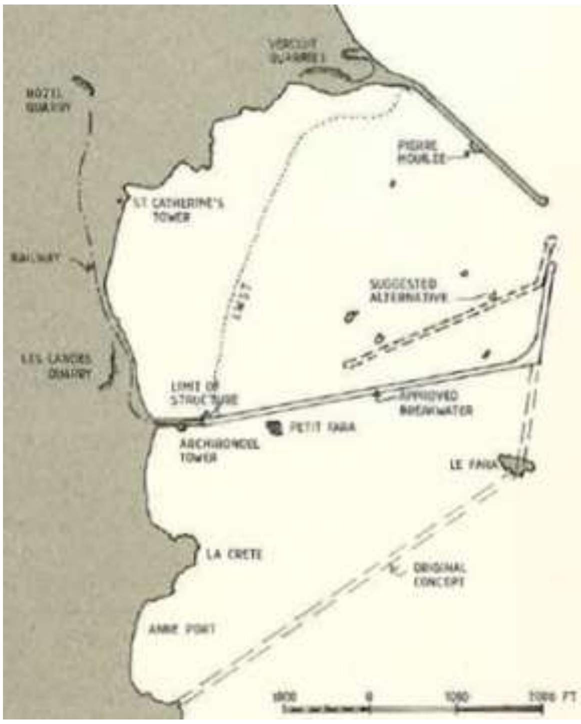


**Figure 12.13 Outline plan of St Catherine's harbour**
*From Davies (1983)*

In the last part of his discussion on Walker, Davies reverts to his root condemnations of the St Catherine's scheme: its remoteness from the centre of trade of Jersey; and the inherent siltation problems of the nascent harbour. He speculates briefly as to whether siltation would have been reduced if the secondary breakwater at Archirondel had been taken further. But he then concludes that St Catherine's harbour would have been useless even if it had been of adequate depth: neither a commercial port, nor a naval harbour, nor a harbour of refuge.

Turning to the construction of the breakwaters at St Catherine's and Alderney, Davies gives a brief summary of the contractor, Jackson & Bean. Thomas Jackson started constructing canals, then moving to railways with Mr Bean, circa 1839. After Bean's death (probably in a riding accident on the Caledonian Canal project), Jackson partnered with Bean's son, Alfred, who later married Jackson's daughter Ann. Jackson did not warm to these two Channel Islands projects, in part because "*he hated the sea*" and that "*he had no practice in constructing breakwaters*", despite which Jackson was clearly the partner responsible for both Alderney and St Catherine's! Davies suggests that the St Catherine's contract was probably commercial successful and was completed relatively quickly. In contrast, construction at Alderney was frequently set back by damage to the plant and to the (partly) completed work, and by the Admiralty's habit of trying to extend the plans. Between 1847 and 1858 five additional layouts were prepared for the harbour, despite the sea having already demonstrated that the designed section was inadequate! The Alderney project "*plodded on and on, apparently indefinitely with a repetitive sequence of building, storm damage and repair, not to mention continuing costs*".

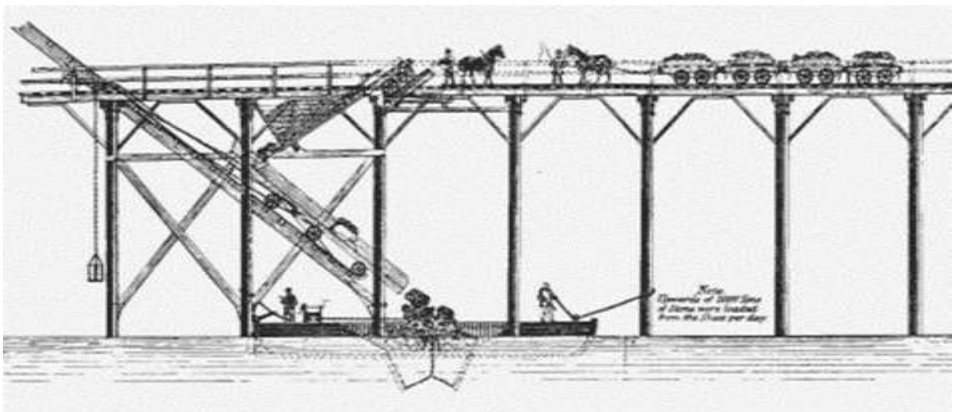

**Figure 12.14 Alderney - Delivery of stone to barges by Jackson & Bean's shoot**
*From The Industrial Railway Record No. 52 - p170-173*

After a long discussion on the difficulties in being paid, Davies highlights a few details of the construction difficulties, and of the plant used in the construction. He notes that a railway had to be constructed to take stone from the quarry to the breakwater, a steam vessel procured for communication / transport, tugs and barges.

They also designed a stone 'shoot' (Figure 12.14) to convey rock from the supply / construction trestle to the bottom-dumping barges.

These devices handled 2000-3000 tons daily. Barges were towed out by steam tugs, and the bottom-dumping operation (up to 100 ton) could be activated on the move "*without the stoppage of the steam-tug*". No details of the operation at St Catherine's are given, perhaps were never published.

At St Catherine's, the tidal range was rather greater, but water depths (at Low Water) were comparatively less. Rock for the rubble mound base was dropped from the construction direct, working outwards ahead of blockwork construction, Figure 12.15. Initially both breakwaters, St Catherine's and Archirondel, proceeded at the same time, although start of construction of the later was delayed by the need to find a better quality rock source, necessitating its own railway. At Verclut quarry, rock for the St Catherine's breakwater appears to have been hauled directly from quarry to construction trestle by horses. Davies makes no mention of steam locos, indeed the list of ~350-400 workers places a blank '–' against the space for "*enginemen and mechanics*". To the south, on the Archirondel breakwater, Walker instructed Jackson & Bean to stop work in July 1849, 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 as forecast by Martin White. In 1851, Walker produced a "*less extensive design for the Southern Breakwater*", it appears primarily to save cost. In Parliament, MPs Joseph Hume, Richard Cobden and William Williams pushed (again) for cancellation of the scheme.[133]

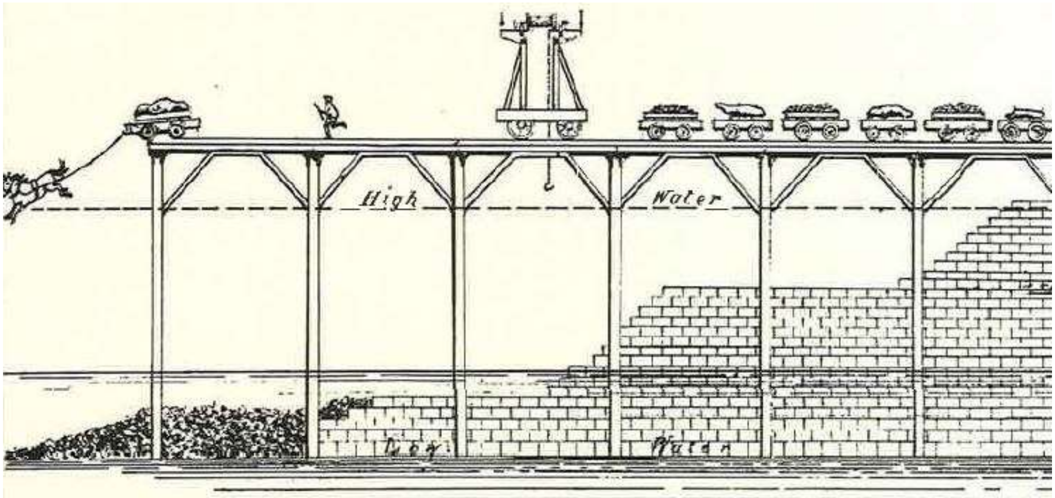


**Figure 12.15 St Catherine's - Delivery of stone to foundation**
*After Davies (1983)*

With the cessation of work on the southern breakwater, progress on the northern arm accelerated, so that the mound was very nearly complete at 2300ft (701m) by 1854 with about ½ of the superstructure to be completed. This was reported to be finished by the last quarter of 1855 with the lighthouse operational in spring 1856. Davies quotes Walker's reports of the total cost as £234,236, exclusive of the costs of land. But without a southern arm to resist south-easterly gales, and with reducing depths, the harbour was almost entirely useless.

Having reviewed the parliamentary debates and votes to fund these works, and admitting the benefits of hindsight, Davies concludes that "*there is little doubt that Government did attach more importance to the harbours in Jersey and Alderney as naval strongpoints rather than simply as havens of shelter*." By 1866, the problem of the essential uselessness of St Catherine's breakwater had been handed to the Board of Trade (Harbours and Lighthouses etc. Department) whose Captain Bedford commented: "*it is anything but agreeable to take up and deal with the cast off works of another Department – cast off too because they can find no use*

133 The original design had the southern breakwater more than 2.5 times the length of the northern (St Catherine's) breakwater.

*for them.*" Within Whitehall, there were various attempts to shift the problem back to the War Office, across to the Home Office, and back to the Board of Trade. Their best option was to pass the problem back to the States of Jersey, but they were reluctant to take on the inevitable maintenance liability. This idea was clouded by various (daft at best) ideas to dismantle the structure (probably just the superstructure) and use the materials so gleaned for breakwaters at St Helier, (or worse) at Bouley Bay. The stand-off between the States and Board of Trade continued to February 1876 when the States Chamber passed a proposition to accept the breakwater, together with a 'dowry' of sufficient land that could produce a net annual revenue of at least one hundred pounds (to balance the anticipated maintenance liability). The subsequent negotiations with HM Receiver-General were concluded by the St Catherine's Harbour Act in December 1877.

## 12.15 Peterhead by Buchan (1984)

Designs for Scotland's only 'harbour of refuge' were prepared during the 1880s by D & T Stevenson (1884) and then by Sir John Coode (1888), and were possibly one of the few genuine 'harbours of refuge' – see especially discussion by Davies (1983) on St Catherine’s (Jersey) and Alderney, and Allsop (2020) on Harbours of Refuge. The Stevenson design envisaged north and south breakwaters with an island one in the centre of the bay. The breakwater section would have been a mound up to -6mLW. Concrete blocks of 50-100t were to have been used to bring the mound up to LW, above which in-situ concrete would form the superstructure wall to +6.7mLW. Around 400,000m$^3$ of rubble would have formed the mounds, and 420,000t of concrete the walls. Construction was anticipated as taking 23 years.


**Figure 12.16 Peterhead South Breakwater construction around 1892**
*Courtesy of Peterhead Bay Authority*

The Stevenson proposal was however rejected, and the alternative by Coode was adopted. Using Stevenson's 'fetch' formula, a wave height of 9.75m might have been anticipated (to some extent confirmed by observations of waves of 8m in the bay in 1888). The Coode design envisaged a mound to -9mLW with a vertical wall formed by concrete blocks of 25-50t. Working from the south side of the bay, the first 108m were formed by a wall of in-situ concrete. This was topped by granite paving, and crane rails for the block-setting Titan crane, all completed by middle 1889. It then took another three years for the two 50t capacity Titan cranes to be fabricated and delivered from Stothert & Pitt to a design by Sir William Matthews, erected, and made operational, during which time the 'barge' harbour was constructed, again with walls using mass concrete with ashlar facing, Figure 12.16

The next 174m of the South Breakwater were founded directly onto rock, mostly using mass concrete in frames to raise to level, placed by bottom-opening bags or skips, occasionally supplemented by large hessian bags filled by concrete. The foundation initially at -9mLW was up to 4.4m thick, later (from 1897) lowered to -13mLW. The main superstructure was formed by concrete blocks, jointed within courses by concrete joggles. Outer block included ashlar facing.

Steam locomotives hauled side-tipping wagons. A second steam Titan crane was used on each breakwater from 1892, and a steam powered barge was added in March 1896. From then to 1911, an average of 23,800t per annum of rubble was deposited to form the (deeper water) foundations.

The breakwaters experienced damage (block displacement) in storms in 1897 and 1898, so from 1900 the mound crest (seaward wall toe) was formed by 40t concrete blocks placed as pitching, and in 1902 the wall width was increased from 14m to 17m. The South breakwater (872m) was completed in 1914, but the North breakwater was not completed (450m) until 1956.

Courses of blocks to a combined weight of 3,300t were moved bodily in winter 1898, although Buchan gives no more details. He does however observe that toe blocks have moved, and that (by 1984) vertical joints had opened in the South Breakwater under settlement (apparently) caused by foundation movement.

### Discussion to the paper

**Mr A M Robertson** noted the disparity between the North and South breakwaters. He notes damage to the 'scar' end during construction of the North Breakwater, requiring use of tie rods to hold the blocks together. He noted damage that occurred to the running end left during the Second World War. He also notes failure of some concrete blocks by disintegration, and damage to the deck, perhaps by overtopping down-fall pressures.

**Mr J M Leonard (Coode & Partners)** noted storms in 1898 with waves > 10m which moved 3,300 tons by 50mm, and in 1900 when waves of over 12m moved a mass of 31,500 tons by more than 125mm. He comments on the use of sloping or 'slice' blockwork, particularly to accommodate early settlement. Leonard also comments that the cost inflation (£746,000 estimate in 1886 vs. outturn by 1958 of £2,000,000), represented moderate inflation.

**Buchan** however counters that the wholesale price index (WPI) in 1914 was very similar to that in 1881, implying no significant inflation. Yet by 1941 when 366m of breakwater were still to be constructed (28%), expenditure had exceeded £793,000, already above the original budget.

## 12.16 Aberdeen by Turner (1986)[134]

This book presents the history of the evolution of Aberdeen harbour. Earliest written record of a harbour here was charter of King David granted to Bishop Nectan to levy charges on shipping (June 1136). Main early development was as a tidal river port. Town Council invited John Smeaton in 1769 to *'investigate the harbour entrance and to suggest remedial measures to create greater depth at the bar'*. Smeaton's report in 1770 (text given in full as Appendix 1) recommended construction of a pier to the north side of the river entrance to hold the Sandness. The main purpose of the North Pier was therefore to limit incursion of sand / shingle into the river / harbour channel, and to train the freshwater outflows. Enabling powers agreed in April 1773, leading to the successful construction of 366m of the present North Pier.

The reduction of the bar in the harbour entrance, and any wave reflections from the inside of the North Pier, did now appear to trap incoming easterly swell *'with a disturbing effect on the shipping lying at anchor'*. Smeaton '*freely admitted that he had not forseen that the greatly increased depth … would also allow greater freedom for easterly swell to enter the harbour*'. Smeaton's second report in 1778 recommended an additional spur jetty or catch pier to act as a wave trap, '*built shortly afterwards*', although Turner does not indicate where.

John Rennie was engaged in 1797 to advise on further improvements to the harbour. The cost of the works that he proposed were however judged to be 'daunting' and the advantages so 'patently limited'. After more debate, Thomas Telford was instructed to draw up a plan for harbour improvements. Telford reported in April 1802. A further report in August 1809 allowed for construction of an enclosed dock, access locks, construction of quays, and redirecting the main flow of the river, extending north and south piers. A succession of Acts in May 1810, June 1813 and May 1829 facilitated these and subsequent works.

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[134] This book has some information on the timing of breakwater construction, but nothing on the details. It is poorly edited with noticeable repetition, but in which the story changes slightly. Surprisingly few source references are cited.

During 1810-11 (within 15 months of passing the 1810 Act), the North Pier was extended to Telford's design by a further 91m, but '*enormous difficulties were encountered' in extending the further* 183m*. A further breakwater of some* 244m *was constructed on the south side of the channel, again with "some difficulties … in the construction*'.

It appears that the City Council was then enmeshed in much criticism of Telford's works, but in 1828 they applied for a renewal of the Harbour Act to permit further works. This (6$^{th}$) Act was approved in May 1829, permitting further quay construction, and channel deepening. Over (probably) the next 10 years, James Walker revised plans by Telford for various 'wet docks', and a new sewer. Much new works were concentrated on new quays / docks within the harbour.

Then in 1869-1874, a new South Breakwater was constructed, and the third (and final) extension of the North Breakwater was built 1874-1879. Turner does not make it clear who was the designer, but implies that it was William Dyce Cay (the Harbour Engineer), perhaps with some input or scrutiny by Sir John Hawkshaw and James Abernethy. See ICE paper by Cay (1874) and the attendant discussions.

1827 – Aberdeen's first steam tug, the *Paul Jones*, built by Alexander Hall & Co, replaced use of the port entrance capstans to haul vessels into the harbour.

1881 – Aberdeen's first steam crane.

## 12.17 Dover by Potter (1990)[135]

First used as a port in AD43 by the Romans, Dover serviced their occupation. Then as one of the Cinque Ports. 1823, the harbour was given to Dover Harbour Board by the Admiralty. Expansion of the port as a naval base, involved construction of three outer breakwater arms between 1898 to 1909, designed by Coode, Son & Matthews and constructed by S. Pearson & Sons. Temporary works used 100 ft of 20 in square timber, each 10 tons (brought from Tasmania). Diving bells, each 35 ton, 17 ftL x 10 ftW (5.2m x 3.05m).

## 12.18 St Catherine's, Jersey by Hold (2009)

This paper gives little history, concentrating mainly on repairs following damage by a storm in December 2005, and almost entirely focussed on the roundhead.

The roundhead wall, a gravity masonry wall with secondary core fill, is founded on a rubble mound from a seabed at approximately -7mCD to mound level at about +1mCD. From this the wall deck is formed to +13.2mCD, with parapet walls extending further to +17.8mCD. The breakwater is exposed to a tidal range on springs of 9.5m.[136]

Prior to the December 2005 storm, differential settlement of deck slabs of 15-20mm, and loss of individual stones from the roundhead face, pointed to distress within the roundhead. The "*significant loss of facing stones*" accelerated in the December 2005 storm, exposing a large void. Movement of remaining facing blocks suggested loss of foundation support. A borehole through the roundhead deck revealed a substantial cavern within the fill between the main masonry walls with an upper void of 10mW x 6mD x 2mH. Below this a further void 2mH was concealed beneath a fallen piece of deck slab. Tests on the fill material "*confirmed that the core material was a poor-quality conglomerate rock that was degrading in the marine environment.*" "*The outer facing blocks were better quality rock and most remained in good condition.*"

Possible consequences of roundhead failure were explored graphically (there is no evidence of any empirical or numerical modelling), and the States of Jersey were persuaded of the need to bolster support to the walls by protecting the rubble mound by large rock armour around the full roundhead to +6.25mCD. initial calculations

[135] The paper is mainly devoted to traffic statistics and changes to the harbour after 1950. See also Wilson (1919)
[136] Perversely, Hold fails to inform the reader of predicted wave conditions given in an HR Wallingford report (EX5255) of February 2006.

suggested that rock up to 50t might be needed, later refined downward to 30t. These were delivered in two barges from Larvik in Norway in autumn 2006. Voids in the wall fill were re-filled using broken slabbing with a geotextile used to retain fines within the core. Facing blocks (2-3t) were dowelled into place using stainless steel fixing pins. Rather than replace the main deck by a rigid slab, an asphaltic wearing course topped granular base and sub-base.

## 12.19 Wick by Paxton (2009) [137]

Development from 1807 of Pultney Town on the south bank of the Wick River for the British Fishery Society was matched by harbour expansion. Telford's old harbour was completed in 1811, then the new harbour 1825-1834 by James Bremner whose South Pier was 9.8m high with a paved revetment (approx. 1:2) curving up to a near-vertical crest. Even so, more capacity was demanded. By 1857 the British Fishery Society proposed substantial expansion with a breakwater projecting north from the south side of the bay at about the -4m contour. A more ambitious proposal by D & T Stevenson was begun in 1863, but damaged in Oct 1868 (75m lost). Then in Feb 1870 the outer 116m was destroyed down to about -1.5m in a 3 day storm.

Paxton judges that Thomas Stevenson would have expected waves of 7-9m. The design was then of a rubble mound to -5.5mLW surmounted by stone block walls, infilled by rubble, with a superstructure width up to 16m wide. Steam locomotives hauled stone from South Head quarries on rails onto staging. Travelling gantries (including 'Jenny' cranes) running on the staging then tipped stone onto the mound, possibly their first use in Scotland. The seaward wall was formed as slice-work battered at 6:1. Below water the blocks were dry-jointed, but used Roman mortar or Portland cement mortar above HW. Paxton claims that the depth to which the blocks were taken was 50% deeper than "the accepted norm". Despite this precaution, intended to avoid impulsive breaking over the mound and onto the wall, the wall was damaged in 1870, then in 1872, and again in 1877. In the latter storm, a block of bonded masonry / concrete of 2640t was moved backwards, and broken into two pieces.

## 12.20 Seawalls in Hangzhou Bay by Wang et al (2012)

This (poorly refereed or edited) paper discusses various aspects of historic seawalls and river walls at Hangzhou Bay.[138] The paper has three main sections. The first describes some aspects of the original construction of the seawall. The second presents some (very incomplete) details of rather simplistic physical model tests on an idealised section of wall. The last section presents an interesting analysis of the original lime mortar in the wall, revealing the use of 5% sticky rice.

Seawall structure created in Ming Dynasty (1368-1644AD) and/or Qing Dynasty (1644-1911AD). Again, the paper is unclear on construction dates. Key seawall segments at Haining (28km, exposed to the river bore up to 5-7m/s), and Haiyan (6km exposed to typhoon waves).

'Fish scale' seawall – surface mud removed to depth of 1.2m below local bed level, timber piles driven into hard foundation until pile crests were level (and below local pre-dredged bed level). First two layers were formed by 5 longitudinal and 5 horizontal (do they mean lateral?) stones. Above this, the number of blocks reduced after each two layers, up to the 14th layer, after which the wall tapered by a block per layer up to the 18th (crown) layer. Above water, seaward face joints were filled by mortar of tung oil[139], hemp and lime. Above 10th layer, blocks were secured together by iron 'buckles'.

[137] The breakwater failure at Wick is discussed in more detail by Allsop & Bruce (2020), and in Chapter 6 of this thesis.

[138] The locations are not clear, so it is not easy to estimate the exposure to wave loadings, but they appear to include the Qiantang river bore, and typhoon waves in Hangzhou Bay.]

[139] Tung oil or China wood oil is a drying oil obtained by pressing the seed from the nut of the tung tree (Vernicia fordii). As a drying oil, tung oil hardens (dries) upon exposure to air.

Model tests to explore response of seawall. Flume 70mL, 1.2mW, water depth 1.0m. Largest wave height 0.4m, period 0.5 to 5s. Froude scale for geometry / waves 1:16.

Approach to seawall toe of 1:8 and 1:6 slope, down to about +0.9m, but then timber pile retained 'step' down of around 1.5-2m to 1:10 river bed slope. A wall height of 5.4m prototype was reproduced as 0.6m in the model.

Test conditions primarily based on the tidal bore wave. But the only results reported were for the 1.5s condition. Highest (model) pressure, $p_{1/3}$ (but NB only regular wave tests). These were then scaled by Froude (145kPa), Allsop *et al*, 1996 (65kPa), and Cuomo *et al*, 2010 (59kPa). They also note suction pressures / forces, but do not quantify them.

The final section of the paper describes an analysis of lime mortar samples found in the lower sections of the seawall, and tests to determine its composition to allow reconstruction, as no written records were ever kept, solely oral communication. Samples analysed by X-ray and electron-microscope methods. The composition was almost entirely Calcite ($Ca(CO3)$), with no sand, but including sticky rice – a known possible ingredient. Mortar reconstruction tests showed increases in strength, and ductility of mortars with up to 5% sticky rice.

# 13 TECHNOLOGIES

## 13.1 Outline

This Chapter extracts from the historical review key technologies bearing on breakwater construction and stability under three main headings:

Construction plant and methods

Vessels (both those using harbours and in constructing harbours)

Cements and concrete

Many of these have also been discussed at length under the General literature in Chapter 11, and Specific Sites in Chapter 12.

## 13.2 Construction plant / methods

### Andrews K & Burroughs S (2011) on Stothert & Pitt

This history of the company that was *'cranemaker to the world'* and '*Bath's greatest contribution to world history*', is illustrated by photos and plates now with the Museum of Bath at Work.

**Table 13.1 Summary chronology for Stothert & Pitt (after Andrews & Burroughs, 2011)**

| Date(s) | Activity |
|---|---|
| 1795 | Stothert registered as Ironmonger and Manufacturer, acting as agent for the Coalbrookdale company |
| 1799 | supplied two cast-iron bridges to span the Kennet and Avon canal using components from Coalbrookdale |
| 1815 | opened the Horse Street foundry |
| 1827 | patented a plough |
| 1836 | set up a separate engineering company to manufacture steam locomotives |
| 1851 | exhibited a hand-powered crane at the Great Exhibition |
| 1857 | opened Newark Foundry |
| 1862 | exhibited a 6t travelling crane at Paris, and won a gold medal |
| 1876 | supplied a 35t crane to patent design by Fairburn to Bristol City Docks |
| 1885 | catalogue included Titan cranes for placing blocks up to 50t, and Messent's patent concrete and mortar mixers |
| 1891 | supplied Titan crane weighing 576t for construction of Peterhead harbour, fed with 50t blocks by an auxiliary Goliath |
| 1893 | supplied electric dockside cranes to Southampton harbour |

### Bartholomew (1870)

The Civil Engineering contributions to the Technical Educator features short sections on breakwaters including a graphic on placing slice-blockwork after Telford at Aberdeen, and a mechanism to ‘float’ out large rocks suspended from wooden casks. Bartholomew introduces breakwater protection to harbours starting at

Alexandria and Pharos. the making and use of concrete blocks. He described hydraulic mortars of silica and caustic lime, and discusses use of pozzolanic materials by the Romans, and of different sources of lime. He then turns to discussing Roman harbours formed of walls with arched opening claiming that this would "*break the force of the waves and (*give) *no risk of silting up* ..." He refers to the "*celebrated breakwater ... at Cherbourg*…" by de Cessart using timber cones showing diagrams of the cones and their flotation by wooden barrels. He describes at some length how these large barrels were attached and released in order to sink the cone in its chosen position. The fate of this construction is however dismissed in two lines: "*the timber of the cones soon went to decay from neglect, and the stones they contained fell into a natural slope* …"

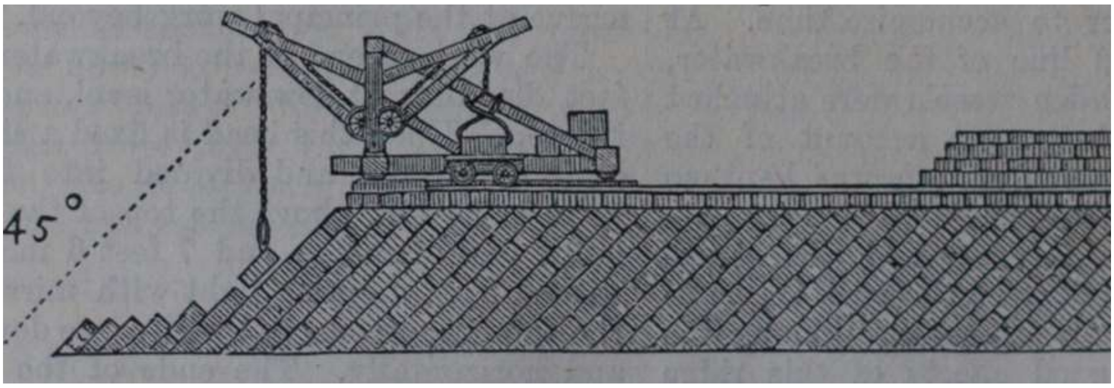


**Figure 13.1 Example of slice-work, inclined blocks**
*From Bartholomew (1870)*

Bartholomew turns to Smeaton's breakwater at Aberdeen completed in 1778. He notes the success in reducing the bar, but also the concomitant effect of allowing larger waves to penetrate the harbour. A "catch-pier" was built on the south side of the channel using rough granite blocks. At the head, stone blocks were shaped with a taper. The header stones were secured by iron cramps "jumped" with wooden wedges rather than leaded into the blocks.

He continues on Aberdeen describing Telford's 1810 extension to the main pier, see graphic on slice-work in Figure 13.1. He particularly describes the mason's methods to split stones along their natural cleavages.

At Wick (here termed Pultney Town) he again refers to timber casks used as flotation. Each cask displaced 445 ft$^3$ (12.6m$^3$), being used in pairs to carry 34t of stone, weighing about 21t submerged, Figure 13.2. The barrels required internal struts to take the compressive forces. Two lift rings were attached to rods each though a drilled hole, held by a wedge. The double lifting eyes prevented the rock from twisting.

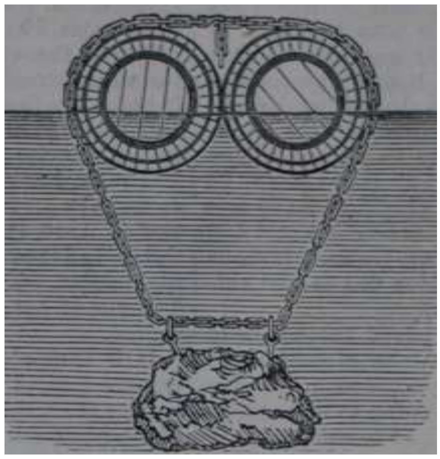

**Figure 13.2 Lifting rock by barrels**
*From Bartholomew (1870)*

Bartholomew then continues with a short description of the breakwaters at Holyhead, and Plymouth where he describes the progress of the contracts, unit prices reducing as the contractors became more familiar with extracting and transporting the rock. Steam tugs were used to tow vessels able to carry 60t, the stones being loaded by rail trucks, discharging by being winched up and tilted from the vessel, itself moored to a locator buoy. At Plymouth (Figure 13.3) he notes that it was two years before stone became visible at low water, so around 1814. Bartholomew notes that the upper slope was formed at 1:3, steeper than Rennie's design, but that the storm of 1824 reduced its slope to his intended 1:5 slope.  The upper part was revised later to increase crest level. By 1841, about 3.4 million tons of stone had been used, at a cost of £1,500,000.

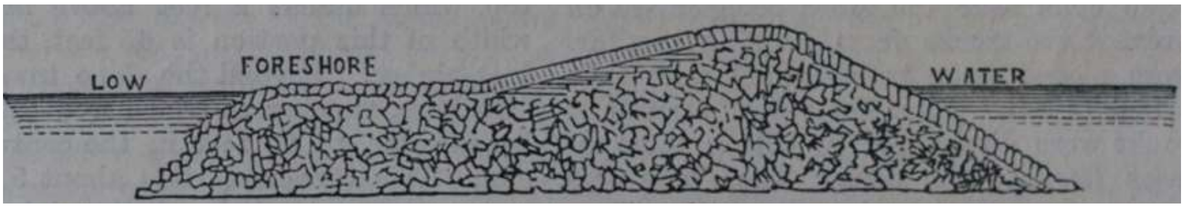


**Figure 13.3 Simplified profile of Plymouth breakwater**
*From Bartholomew (1870)*

Bartholomew refers briefly to the breakwater at Lyme Regis, referring to it as formed as *pierre perdu*, but then turns to the breakwaters at Portland where he pays particular attention to the construction from staging at 18ft above HW (5.5m). He notes that the 15" square piles (0.38m side) were tipped by Mitchell's patent screw, see Figure 13.4. The staging so supported carried lines of rails, "*allowing as much as 2,200t*" of stone to be discharged per day through bottom-dumping rail wagons, each carrying up to 7t. He describes the general form of the rubble mounds, and notes that the upper walls were formed by Portland stone faced by granite blocks.

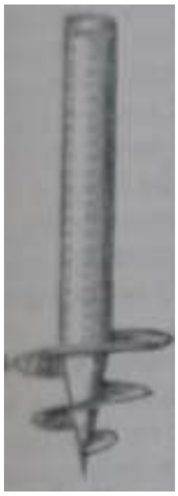

**Figure 13.4 Mitchell screw**
*Bartholomew (1870)*

He then discusses Ramsgate, noting that Smeaton had been invited to advise in 1774. He notes that a wooden pier to the west and stone pier to the east, each "*carried out by separate engineers, or rather amateurs*." Noting the prevalence of this harbour to accrete, he praises Smeaton's use of sluiced flushing from "*an artificial backwater*", noting however that this was a little too effective, leading to scour of the chalk at the breakwater heads! In protecting the seabed around the pier heads, Smeaton made the first use in July 1788 of a 'diving bell', a rectangular iron chest of about 2.5t.

Bartholomew concludes with a discussion on (the futility) of floating and vertical screen breakwaters. He rehearses how floating breakwaters might be expected (by the optimist) to work in damping wave action, but notes the ability of waves to transmit underneath, concluding that "*floating breakwaters are amongst those achievements of engineering which are by no means impossible*" but "*have yet to be accomplished*". Similarly, he considers a vertical plate or screen breakwater, but observes "*the utter hopelessness of securing*" "*a wall of iron*" "*against the attacks of the sea*".

## Kenny (1895) on Machinery for Sea-works

Starts by noting the paper by Pitt (1893) to which he views this paper as an extension. He therefore mainly describes equipment not discussed by Pitt (1893).

Kenny emphasised the need for: meteorological measurements to aid forecasting (of storms); measurements of lengths and depths; of currents; of bed movement. For shallow depths, he used a railway bar, 7.6m long, towed be a bridle from a steam-launch and suspended by a lifting line from a boat towed by the launch. The drag bar will 'catch' on any protruding rock pinnacle. The method had been approved by the Colonial Marine Engineer of New Zealand in 1891. He described concrete-making at Gisborne Harbour (started in 1886) where three Blake-Marsden stone-crushers were powered by a 25HP semi-portable steam engine. Skips were towed by horses, although he believed that their replacement by a steam locomotive would "*have been much more convenient and more economical*". The Goliath crane at Gisborne was purchased from England in 1886. Blocks at Gisborne were placed in conventional horizontal course by a rotating (±22.5°) Hercules crane. At Napier (completed 1886), blocks were set inclined onto a mound at 5.5 to 7.4m below MLWS.

Salt water was used in the concrete, pumped by a Cameron pump. He makes a number of suggestions of improvements to skips and the Lee mixer. Elsewhere in New Zealand, rotating jib cranes had been used to lift blocks of 30-40t. he cites various iron hoper barges used at new Plymouth and Napier to deposit material onto the rubble mound.

For dredging, he mentions a 7" Ball sand-pump, grabs deployed from the cranes, single-chain dredgers, and Lobnitz rock-cutters.

In discussing the drilling of bolt-holes into (quay) walls, he mentions "… *rock-drill … actuated directly by steam from the crane boiler, or by an air-compressor working on the crane … electric, pneumatic or hydraulic transmission …*"[140].

### Kidd (1891) on underwater construction (with concrete)

Kidd describes construction of a pier about 8 miles south of the mouth of the Tees completed in 1891 for the Skinningrove Iron Company (who produced a hydraulic cement using blast-furnace slag from Cleveland). Using that cement, Kidd formed a concrete mound of about 2m height up to about 0.3m above MLWS. A trench was first dredged through the sand using an 8" centrifugal pump with hoses guided by divers, clearing down to the under-lying shale. Freshly-mixed concrete was then placed by hopper boxes holding 0.6m$^3$ at a time, Figure 13.5. The natural side slope formed was about 1:1, later cut away on the harbour side. The top surface was trodden down and levelled by divers. This concrete foundation mound was taken out to 91m length, and it was found possible to place it up to waves of 1-1.2m. Once in place for more than 12 hours, little damage was caused by storms.

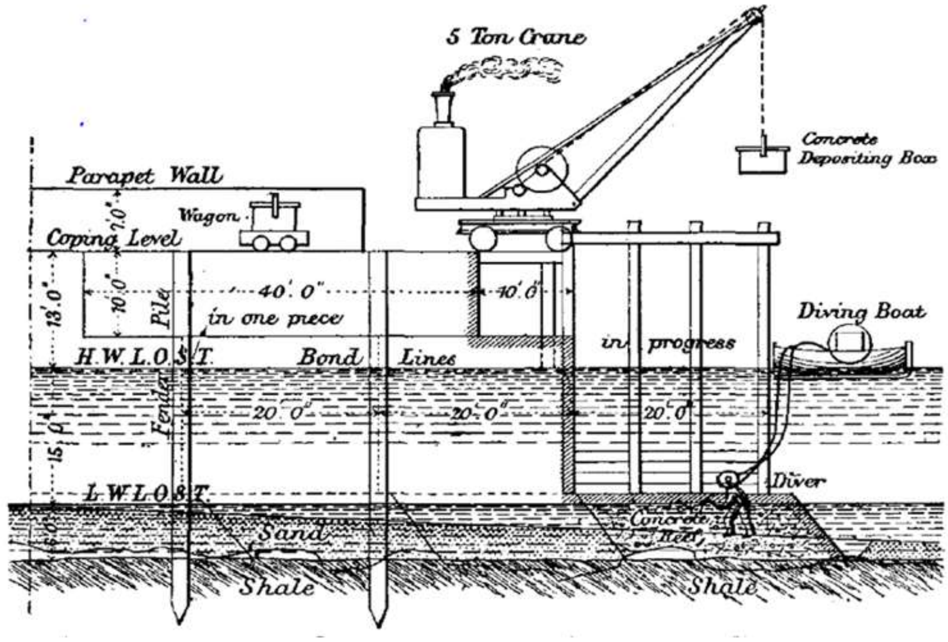


**Figure 13.5 Construction of North Breakwater at Skinningrove**
*From Kidd (1891)*

Once the foundation mound had set hard, timber formwork was erected with a 12:1 batter, held by 25mm tie 'bolts'. The upper wall was also formed by in situ concrete.

[140] Which implies that by 1890 the contractor had available a wide range of possible options for mobile power.

## Pitt W (1893) on Plant for Harbour- and Sea-works[141]

In his preamble, Pitt notes "*... vast development ... has taken place ... is largely due to the introduction of concrete and the consequent evolution of ... methods of construction which depend on its use ...*". He notes how advances in plant have allowed the designer to increase heavier blocks, and at greater reaches, and that therefore designers have demanded greater capacity. The prime example is the Titan block-setting machinery in its various types. But first, the concrete must be mixed. He notes two kinds of mixers: continuous and intermittent (later termed batch) machines, for which he ends to prefer the latter for less "*variation of quality*". In batch mixers, Pitt identifies those devised by Messent, Lee, Ridley, and Punchard. He notes the need for external power sources observing that Sir John Coode had used a portable steam crane to lift materials and power the mixers, steam being carried from the crane's boiler down through the centre of the crane-post.

In the block-yard (already we see the assumption of breakwater and quay walls being composed mainly of concrete blocks), blocks are moved about and stored using Goliath or traveller cranes running on two rails with spans of 15-17m. He cautions that movement of the traveller or 'crab' may lead to oscillations, perhaps in time with the engine motions. He shows three iterations of the general Goliath type for harbours at Karachi, Gisbourne, and Peterhead, each with a steam powered traveller or crab operating within the 17m span. At Peterhead the working load was 50t and the whole assembly weighed 109t. The main machine was carried on six wheels each side, each of cast-iron centres with a rolled steel tyre shrunk on. At each end, two subsidiary wheels at right angles can be lowered to permit the whole assembly to be traversed on a set of cross-rails. Further details are given on the wire-ropes used, the reeving arrangements to avoid twisting, the lifting barrel, and types of brakes used on the barrel. Four-wheel block trucks, also running on rails, may accommodate 40t blocks. For those blockyards using gravity to move these trucks, then powerful brakes must be built into each truck.

The largest machines are usually the block-setting machines, known generally as Titans (originally non-revolving); Hercules (revolving or radiating); Mammoths (two very large machines used at Tynemouth. Pitt however notes that the name Titan has latterly been applied to all these, including revolving machines. He believes that the first Titan was built in 1869 for Manora (Karachi), laying blocks of 27t, and proved up to 40t. For all non-revolving machines, it is essential to be able to pick-up the block from below the crane, and then traverse it outward. For the rotating machines, the pick-up point must be outward of the slewing ring, so may require the whole upper part to slew around for each block movement. He discusses (adjustable) water ballast, but prefers the more recent use of solid ballast to avoid loss of counter-weight.

The first slewing Hercules was made in 1876 for East London, South Africa, and was the largest in existence at the time with a proof load of 30t. The Titan at Peterhead designed by W. Matthews was designed for a working load of 50t, proved to 62.5t, and a radius up to 30m.

Most of the machines discussed seem to have been designed for a single site, although re-assembly and re-use at a second site is mentioned in passing.

Turning to the placement of concrete in bags, Pitt describes various 'bag-boxes' and their operation, some hung from cranes, others running on rails and end or side-tipping. One design by W.C. Punchard could accommodate 70t bags. Smaller skips designed by Murray or Woodford may deposit concrete underwater. Grabs designed by Wild or Peters may be used pick up loose material from the seabed.

[141] It is probable (but not stated) that Walter Pitt is of Stothert & Pitt. His familiarity with crane making and all related matters suggest strongly that he was a descendant of Robert Pitt who was a partner at the founding of Stothert & Pitt in 1844.

## Discussion and correspondence

The discussion started with a short contribution by **Walter Pitt** on a 'cylinder-sinking grab' intended to excavate and remove material to facilitate sinking open-ended cylinders. The device he showed had been designed for (the late) Sir John Coode and used a 10t traveller to weight it to penetrate stiff clay and some rock layers.

**Mr Rapier** (of Ransome & Rapier) emphasised the importance of spring suspension for heavy Titans, without which it was possible to cause extreme damage to the rails. "*He had seen rails coil up before these machines in a single journey … bend up 3 feet …*". He also confirmed the desirability of multiple rollers and plenty of wheels on the undercarriage, all sprung. He emphasised the need to establish the outer radius and the depths to which a Titan might be required to work, needed to establish key dimensions and the size of the winding barrel.

**Mr Matthews** (later Sir William, of Coode, Son and Matthews) also emphasised the need for "*plant of the very best character*". He remarked that the appropriate plant to set blocks of 30t "*for practically the same sum as a 15t block*", implicitly making the case for investing in (larger) plant. He cited particularly the breakwater constructed at Columbo under the supervision of Sir John Coode where blocks up to 33t "*were manipulated with absolute ease …*" resulting in "*… the most rapid execution of a breakwater ever accomplished.*" He strongly supported Mr Rapier on the use of springs in the undercarriage, giving far "*less wear and tear"* than un-sprung wheels.

**Mr Vernon-Harcourt** discussed use of several cranes of different types at Reunion, Dublin, Tynemouth, Holyhead and Peterhead. He preferred the use of the breakwater-mounted Titan constructing end-over-end rather than the use of temporary staging as at Holyhead and Alderney which "*often involved the loss of some staging during the storms of winter*".

**Sir Benjamin Baker** commented that early engineers would be astonished to see the modern methods to deposit blocks. He noted documents in the Public Record Office and at Windsor Castle showing methods used to build the mole at Tangiers (Routh, 1912). He cites work by Mr Shere who "*… in June 1677 … floated out and deposited the first of his great monolithic blocks weighing over 2000t*". (Here Baker has confused himself by omitting the fact that Shere's caissons were hollow timber boxes, floated out from England empty and probably only weighed 2000t once ballasted and filled with sand / rock and tarras – see Routh, 1912.) Baker notes that the Tangier contract had originally been let to the Earl of Teviot, Sir John Lawson and Sir Hugh Cholmley in November 1662. After various cost increases, King Charles II cancelled the contract and appointed the contractor's agent, Mr Shere.

On grabs, Baker recalled a paper read at ICE about 25 years previously (so around 1868) on the Millroy excavator. He also noted a machine invented by M. Gouffe in 1703, and published by the Academy of Sciences.

**Mr Charles Walker** made some remarks on concrete-mixing machines. He noted that Messent machines were now more than 20 years old. He had used a Lee machine at Hartlepool which worked well. He preferred fixed mixers for block-making, rather than mobile machines. On grabs, for excavations in London clay, he had good experience with Bruce diggers without guides in the cylinders. He sank about 200 cylinders (piles) at Limehouse around 1875 at a rate of about 2 ft / hour.

**Mr PK Stothert** (also of Stothert & Pitt) made further points on large cranes, but particularly emphasised the importance of effective brakes on the winding barrel, regarding "*… the lowering … being about 90% of the work*"[142], but making the point. He advocated the Matthews Hydraulic Brake.

[142] Perhaps a slight over-estimate

**Mr EA Cowper** mentioned the work of Bindon Blood Stoney at Dublin lifting / lowering 350t blocks from floating plant (Stoney, 1874).

**Mr EE Sawyer** noted use by Sir John Hawkshaw of a revolving Titan at Ijmuiden (before or shortly after 1869?). he debated the merits (and costs) of rectangular vs. rotating Titans, citing the need to place wave-breaker blocks along the toe of the wall as a reason to require a rotating crane. He particularly cited the Titan used at Columbo to which he gave a cost of £7,500 (perhaps equivalent now to £700,000).

In the Correspondence, **Mr CJ Appleby** had comments on various machines "*differing from those described by the Author*". He particularly doubted the rate of concrete mixing of 120t per mixer per hour implied in earlier comments. His main emphasis was on sites that did not merit the high investment in plant. He touched on harbour-works at Las Palmas where the annual grant available would not support machinery or staff being run all year, so they tended to use manual labour rather than steam power. Materials were moved around on 2ft gauge rails, demounted when not needed. Within the blockyard, trucks moved by gravity, "*so that from first to last the loads were always on a down-gradient.*" Hand-gears were fitted to start the trucks, gravity did the rest. Appleby also refers to concrete making for Batavia Harbour (probably now known as Jakarta, Indonesia) where Lee-type mixers were driven (and towed around) by a steam crane. Dry materials were gauged by volume.

**Mr AE Carey** "*thought the scope of the paper very limited"* with the remarks on concrete-making being confined to three types of machine. He noted use of a portable Carey-Latham machine used at La Guaria harbour, Venezuela, to produce liquid concrete for sack-blocks in sizes of 70t, 130t and 160t, Figure 13.6

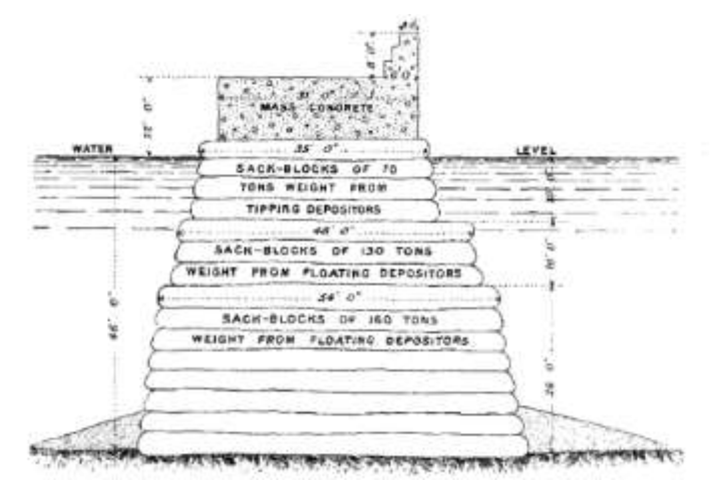


**Figure 13.6 La Guaria Breakwater**
*From discussion by Carey to Pitt (1893)*

**Mr Dyce Cay** had used intermittent and continuous mixers at Arbroath and generally preferred the latter. He emphasised advantages in use of timber rather than iron for staging, particularly where the timbers could later be re-used in other parts of the works. He noted that use of blockwork placed by a Titan required that the breakwater wall must be capable of carrying the Titan. In contrast, he preferred to construct using liquid concrete in bags or in mass behind formwork as at North Pier Aberdeen, and at Fraserburgh. He also mentioned the construction at Lerwick in 1883 using floating plant.

**Messrs Easton and Anderson** described plant used at the Port of Poti on the Black Sea in 1875. Four Goliaths were used in the block-yard. Two timber framed radial Titans were used to set blocks up to 20t, each driven by a locomotive boiler and 10t steam winch mounted on the crane.

**Sir Charles Hartley** commented on construction using a Titan at Leixoes, Portugal, which he had visited in 1888. He gave more details of this machine, with an engraving. It was used to place 50t blocks of rubble in Portland cement, and granite blocks up to 8t loaded into wagons to be tipped into the sea at up to 750m$^3$ per day. He also gave details of block handling at Sulina on the Danube supplied by British manufacturers.

In contrast to these blockwork breakwaters, he noted the technique of concrete blocks "*thrown down at random*" used by Mr Poirel at Algiers in 1840, and at Marseilles, Port Said, Biarritz and Cette. This required more modest plant, simple decked barges and floating derricks being sufficient. At Marseilles, 20,000m$^3$ of stone and 4,000m$^3$ of concrete blocks were deposited by these methods per month. In Italy before 1850, pozzolanic concrete was retained by "*planked dams*" (timber formwork?). He noted that "*greatest merit would always attach to the engineer who selected the means of construction best suited to … the work to be done*."

**Mr Edward Jackson** felt that the major advances were due to the knowledge and use of concrete, and thereby to the development of the Titan, which he felt offered "*least possible risk and uncertainty*". He discussed grabs, and described use of the Bruce-Batho grab in 1879.

**Mr Redfern Kelly** discussed placement of underwater concrete from skips. He noted that the provision of upper doors to the skips reduced cement wash-out prior to the placement in situ. Strict supervision was needed to avoid concrete skips being lowered too rapidly, thus disrupting the mixture, and to avoid placement in conical heaps rather than the even layers desired.

**Mr W. Kidd** noted that many projects were too small to support use of equipment of the sizes discussed in the paper. He had previously discussed (Paper 2542 in 1891) use of small jib cranes to place concrete in mass.

**Mr I.J. Mann** noted the discussions on breakwaters composed fully of concrete blocks, and enquired as to whether block-setting cranes had ever been used to form conventional piers with two blockwork walls with fill between? He described briefly use of floating barges to place 100t blocks at Brest using the tidal range to lift and lower the blocks.

**Mr W.H. Price** wished to clarify the history of the development of Titans. That at Karachi was designed (and named) by himself and the late Mr W. Parkes in July 1868. It was built by Stothert & Pitt in 1869. The Alderney 'Sampson' discussed by Vernon-Harcourt was a relatively small wooden rotating crane with a long jib, designed for staging piles, not 27t blocks. The Titan crane devised by Hutton, Hawkshaw and Hayter had been developed independently. He credited the use of sloping blocks to the late Mr W. Parkes.

**Mr William Shield** (of Peterhead) felt that use of staging and/or floating plant would continue alongside the development of those devices covered by the paper. He noted the omission of the Carey-Latham continuous mixer, used at Newhaven to fill 100t bags in about 20 minutes. He described use of a similar machine at Peterhead mixing 20 $yd^3$ per hour (15.3$m^3$/hour). He also described a 'home-made' Goliath used at Algoa Bay around 1883 formed by "*borrowed*" material, then dismantled and re-absorbed into the works.

**Mr Bindon B. Stoney** requested the Author to give the cost of plant. He cited equipment described in his previous paper (Stoney, 1874) which "*possessed the advantage of not being subject to a patent*."

**Mr Walter Pitt** responded to the correspondence. The successful use of continuous mixers depended critically on "*proper handling of the materials before they passed into the mixer*". He noted that the block-setting machine Titan at Leixoes was built by the Fives-Lille Company.

## Vanderkiste W (1846) Machinery for working a diving bell at Kilrush

"*The old pier at Kilrush … did not afford sufficient accommodation for the steam vessels …*", which led to the need to extend the pier into the sea. Bed level averaged about 8ft (2.4m) below CD, so needed a diving bell to form the foundation and bed the pier masonry. Timber piles (0.3m square) were driven 2-3m into the clay and gravel bed at 6m spacing. The pile tops were cut level to receive beams and rails to carry a travelling frame with crab winch and air pump (both hand-powered). Another travelling frame was used to lower the dressed stone. The bed had been levelled by crowbar and picks from within the diving bell.

The bell was of cast iron, weighing 7t, displacing around 5t of water, so weighing 2t when submerged. The air supply was pumped by three men, the crab was worked by two men, two further winched the travelling frame, and (usually) two in the bell.

## 13.3 Diving helmet

Charles Deane (1796-1848), invented and patented a Smoke Helmet in 1823. Later (1828) he and his brother converted the concept to a diving helmet. By 1836 the Deane brothers had used the helmet in various salvage operations, and had produced the first diving manual.

Augustus Siebe (1788-1872) improved and commercialised the helmet, leading to production from about 1837, with the firm Siebe & Gorman being founded in 1870.

## 13.4 Diving bells

Various forms of diving bells have been known since the 4th century BC. From early / mid 1600s, diving bells had been used for salvage operations, and could presumably have been made available for engineering works, if resources permitted.

The French inventor of the pressure cooker, Denis Papin improved air pumping into a diving bell in 1689. This design may later have been used by John Smeaton in 1788 who incorporated several improvements to the diving bell, including the first efficient hand-operated pump to sustain the air supply via a hose; an air reservoir and non-return valves to keep air from being pushed back up the hoses when the pump stops. The design was updated to be used underwater on the breakwater at Ramsgate.

## 13.5 Vessels

### Barnes (2014) on ship engines

Published by IMarEST, this book is in two parts. The first, chapters 1-21 reviews the development of ships engines from the *Palmipede* in 1774, via the *Charlotte Dundas* in 1803 to LNG fuel or nuclear. Part 2, chapters 22-27 covers the history and activities of IMarEST.

Appendix 1 summarises significant dates in the evolution of marine propulsion from which this table is (partly) drawn. [Minor additions on *Gloire*, *Warrior* and *Devastation* from Wikipedia.]

**Table 13.2 Key steps in marine propulsion**

| Dates | Activity |
|---|---|
| 1736 | Jonathan Hulls patents and submits design for steam engine powered vessel |
| 1774 | Marquis de Jouffroy *Palmipede* paddle steamer on the River Doubs |
| 1783 | Marquis de Jouffroy *Pyroscaphe* paddle steamer on the River Saone |
| 1802 | Symington's *Charlotte Dundas* steams on the Forth & Clyde Canal, towed two 70 ton barges |
| 1807 | *Clermont* using Boulton & Paul engines on the Hudson River |
| 1808 | First seagoing voyage by paddle steamer *Albany* from Hudson River to Delaware River |
| 1815 | *Margery* by William Denny, Dumbarton, first vessel to steam on River Thames |
| 1818 | Paddle steamer *London Engineer* by Henry Maudslay with 120hp engine plying between London and Margate |
| 1819 | Paddle steamer *Savannah* makes first ocean voyage in 23 days |
| 1820 | Royal Navy vessel *Monkey* built |

| | |
|---|---|
| 1828 | Multiple steam powered vessels listed in the Official Navy List. |
| 1829 | Josef Ressel tests screw propeller on steam vessel |
| 1836 | Francis Pettit Smith demonstrates new propeller |
| 1838 | *Sirius*, first vessel to cross Atlantic by steam power alone |
| 1840 | Around 70 further steam powered vessels added to Royal Navy. |
| 1843 | *Great Britain* built at Bristol fitted with screw propeller |
| 1845 | Trial between HMS *Rattler* (propeller) and HMS *Alecto* (paddles) won by *Rattler* |
| 1859 | *Great Eastern* enters service<br>[Launch of French warship *Gloire*, steam powered, with sails, iron-clad so outclassed the British timber-hulled ships.] |
| 1860 | HMS *Warrior* launched - first iron-hulled warship, steam powered, but still with sails |
| 1871 | HMS *Devastation*, first fully steam powered, no sails, so turretted guns on deck rather than below. Twin propellors, two stage trunk engine. |
| 1876 | HMS *Inflexible*, first large ship with vertical compound steam engines |

## Steam powered vessels – adapted from Wikipedia

In 1736, Jonathan Hulls was granted a patent in England for a Newcomen engine-powered steamboat (using a pulley instead of a beam, and a pawl and ratchet to obtain rotary motion). William Henry of Lancaster, Pennsylvania, learned of Watt's engine on a visit to England, made his own engine which he put in a boat in 1763. The boat sank, and while Henry made an improved model, he did not appear to have any significant success, though he may have inspired others.

In France, by 1774 Marquis Claude de Jouffroy and his colleagues had made a 13-metre steamboat with paddles, the Palmipède. The ship sailed on the Doubs River in June and July 1776, apparently the first steamship to sail successfully. In 1783 a new paddle steamer, Pyroscaphe, successfully steamed up the river Saône for fifteen minutes before the engine failed. In the USA, James Rumsey built a pump-driven (water jet) boat and steamed upstream on the Potomac River in 1786.

In 1788, a steamboat built by John Fitch operated in regular commercial service along the Delaware river between Philadelphia PA and Burlington NJ, carrying as many as 30 passengers. This boat could typically make 7 to 8 miles per hour, and traveled more than 2,000 miles (3,200 km). The Fitch steamboat was not however a commercial success, as the route was adequately covered by wagon roads. The following year a second boat made 30 miles (48 km) excursions, and in 1790 a third boat ran a series of trials on the Delaware River before patent disputes dissuaded Fitch from continuing.

In UK, William Symington had built steam pumping engines and mill engines. In 1793 he developed a pivoted crosshead beam above the vertical cylinder to transmit power to a crank. He also worked on experimental steam-powered paddle boats with Captain John Schank. A meeting of directors of the Forth and Clyde Canal Company in June 1800 approved his proposals on the basis of *"...a model of a boat by Captain Schank to be worked by a steam engine by Mr Symington"*. The boat was built by Alexander Hart at Grangemouth to Symington's design with a vertical cylinder engine and crosshead transmitting power through a crank to the paddlewheels. Trials of the first design on the River Carron in June 1801 were successful and apparently included towing barges from the River Forth and along the Forth and Clyde Canal.

In 1801 Symington patented a horizontal steam engine directly linked to a crank, driving a large paddle wheel in a central well in the hull, and got the support of Lord Dundas for a second steamboat named after Lord Dundas's daughter, Charlotte. The wooden hulled vessel was 56 ft (17.1 m) long, 18 ft (5.5 m) wide and 8 ft (2.4 m) depth, Figure 13.7. The boat was built by John Allan, and the engine by the Carron Company. The first sailing was on the canal in Glasgow on 4 January 1803, with Lord Dundas and relatives / friends on board. In March 1803 the Charlotte Dundas towed two 70t barges 30 km along the Forth and Clyde Canal despite "*a strong breeze right ahead*" at an average speed of about 3 km/h (2 mph). This demonstrated the practicality of steam power for towing boats. Opposition by canal authorities however thwarted plans to build more such vessels and the *Charlotte Dundas* was left in a backwater until broken up in 1861.

Name: Charlotte Dundas

Builder: John Allan

Maiden voyage: 4 January 1803

Fate: Broken up, 1861

Length: 56 ft (17 m)

Beam: 18 ft (5.5 m)

Depth of hold: 8 ft (2.4 m)

Propulsion: Carron Company steam engine, Single central paddle wheel

Speed: 2 mph (3.2 km/h)

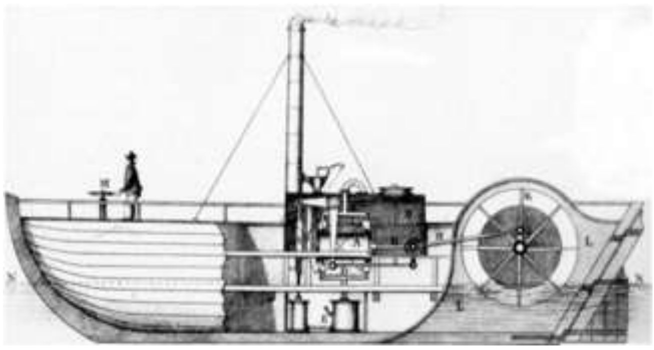

**Figure 13.7 Charlotte Dundas**
*Courtesy Wikipedia*

Although plans for the Forth and Clyde canal were thwarted by fears of erosion of the banks, development was taken up elsewhere, including Robert Fulton's North River Steamboat of 1807 and Henry Bell's PS Comet of 1812. The first sea-going steamboat in Europe was Richard Wright's first steamboat "Experiment", which steamed from Leeds to Yarmouth in July 1813. In 1817, P.S."Tug", was launched by Woods Brothers, Port Glasgow, and became the first steamboat to travel round the North of Scotland to the East Coast.

## 13.6 Cement and concrete

### Bernays (1880) on Portland Cement Concrete pp87-97

The paper relays the author's experience in constructing an extension to the Royal Dockyard at Chatham. In doing so, he puts the growth of use of OPC into context "*... less than twenty years ago, before the use of Portland cement concrete had become general...'* So that dates the '*general*' use of OPC from about 1860 or so, although he then points to "*... a first attempt ... to construct engineering works wholly of concrete ... about the year 1835, built a graving dock at Woolwich dockyard and sea walls ... entirely of that material. The concrete was made of ... hydraulic lime, and of Thames or beach gravel, and was mixed with hot water.*" He notes that the dock floor "*later blew up from weakness of construction*", (perhaps in truth a failure of geotechnical design), "*but the sea wall at Woolwich is standing to this day ... in remarkably good condition*". Yet when the contract for extending Chatham dockyard was put out for tender in 1867, there was no mention of Portland cement. Indeed pozzolano and Roman cement were specified for all work below High Water. He comments that Portland cement "*... with two excellent manufactories within a few hundred yards of the works, was wholly ignored.*". Having tendered on the basis of using blue lias lime, the contractor then found it "*... difficult to secure steady ... supplies ... still more difficulty to secure its thoroughly fine grinding ... so it was determined to substitute Portland cement for blue lias lime in the preparation of concrete...*". In doing so, however, they used a remarkably lean mix of 12 parts ballast to 1 part cement versus the original specification of 6 parts river ballast to 1 part blue lias lime. [It may be worth noting that Cay (1874) used 6 parts aggregate to 1 part cement; and Carey (1886) used 7 parts aggregate to 1 part cement. Further, examination of the drawings at the end of Bernays' paper identifies a number of classes of concrete, all in the ratio 1 part of cement to:

| | |
|---|---|
| Common concrete for building blocks | 10 parts gravel; |
| Face concrete for blocks | 4 parts gravel; |
| Hard face concrete | 2 parts sharp sand, 4 parts slag or broken rock; |
| Bedding concrete | 8 parts gravel; |
| Surface paving facing | 2 parts of pea gravel (no sand)] |

Tests at Chatham were conducted on the neat cement cast in 1.5inch by 1.5inch blocks and broken at 7 days. Around 50,000 tons of cement were used on the project with prices varying from £1.4 to £1.95 per ton (1880 prices). He notes that, once accepted, cement was stored in bulk in bins in a closed shed up to a depth of 4ft (1.25m) for a minimum of 3 weeks to allow "*air-slacking of all free lime in the cement*", it being " ... *unsafe to use any cement until this process has been effected*". The author notes that this exposure to air increases its bulk by up to 5% without increasing weight, and that cement was usually mixed by volume, not weight. In use, they bagged the cement in direct proportion (1/12) to the volume of shingle in the gauge box, avoiding "... *carelessness or fraud in mixing the concrete.*"

At this site, mixing was always done by hand, despite the availability of mixing machines, citing difficulties in moving the machines to the spot where the concrete was needed. Here Bernays offends substantially against modern thinking in advocating that the concrete is "... *made thoroughly wet ... an excess of water is beneficial rather than injurious to the concrete.*"

He notices that other "… engineers have found that large masses of concrete show horizontal and vertical shrinkage cracks on the exposed surfaces", but that such cracks had seldom occurred in the works at Chatham. He identifies three possible contributions: "*free use of water*"; "*aerating the cement*", and restricting layers to 18 inch thickness (0.45m). In limiting heat of hydration effects, the restriction on layer thickness was probably seminal.

For the "*hard face concrete*", a mixture was used of 4 parts crushed iron ore slag : 2 parts coarse sharp river sand : 1 part Portland cement. A temporary removable shutter was used to separate this face concrete from the much leaner mix of the main bulk during pouring and initial compaction, then being withdrawn to allow the two materials to be compacted against/into each other.

For steps and paving, he recommends omitting any sand, using a lot more cement in a mix of 2 parts aggregate to 1 part cement. For use in copings, "... *it is found that, if dressed over with a diamond-pointed hammer, the foot-hold is much improved ...*". (One assumes that refers to the shape of the hammer head, rather than the inclusion of diamonds per se.)

Bernays then finished with breakdowns of the costs of the various types of concrete used on this site, costed using "*ordinary London rates for material and labour*".

### Grant (1880) on Portland cement, pp98-179[143]

Grant noted significant change in manufacture and use of Portland cement over the previous 20 years, particularly in tests / experiments from 1858, and in his papers to ICE in 1865 and 1871. Grant notes the essential requirement for precision in preparing test samples to allow results to be compared. He particularly

[143] This paper was given in concert with that by Bernays (1880).

noted that most public works in Austria and Germany were under the auspices of government engineers using agreed rules for specifying and testing cement, from official testing establishments. He advocates the German procedure of testing sand / cement briquettes (3 sand : 1 cement) tested at 28 days, much preferring it to the testing of neat cement briquettes.

Grant notes materials used in cements: clay and chalk from the Thames and Medway, Boulogne (France) and Stettin (Germany); clay and limestone from Rugby, Stockton, Bridgewater, Poole and Wareham; and marl and limestone from Biebrich, Mayenne and Bonn. He describes the initial processing, then firing or 'burning' using either coal or coke, noting particularly use of the Hoffman kiln in use in Germany, and by the Pluckley Brick company. He discusses "*heavily-burnt"* versus "*lightly-burnt*" cements, although he does not identify these states much further. He then discusses grinding, noting the German and Austrian requirement for less than 20% retained by a mesh of 3 per mm (perhaps 200micron). In contrast English practice might be for 15-25% retained by a mesh of 2 per mm, somewhat finer.

His discussion on strength, rate of setting, and degree of 'burning' is somewhat confused, but concludes that finely-ground cements are generally superior. He follows with a wide summary of properties and tests, including many by Dyckerhoff of Biebrich. He notes particularly tests showing that the tensile strength of 3:1 sand / cement briquettes was ~1/20 the compressive strength.

Grant notes the versatility of concrete, and the potential for special mixes for special uses, as described by Bernays, and its successful use in a wide range of applications.

## Discussion to Bernays (1880), Grant (1880), pp180-208

**GF White** (of JB White & Brothers Cement works) referred to a paper by General Scott and Mr Redgrave. He discussed different views on the permissible levels of lime in cement, distinguishing between use of cements for e.g stucco versus engineering purposes. He then discussed methods of preparing the clay and chalk before drying, how to dry it, and particularly the use of 'surplus' heat from the kilns. He discussed use of Hoffman kilns, but contrasted cheap fuel and high labour costs in UK versus the converse on the Continent. His main argument was that: "*... it would be desirable if the engineers of this country could agree on some uniform conditions of testing cement ... scarcely any two engineers made the same requirements... one relied on fineness ... another a certain tensile strength ... while a third ... a high specific gravity ... though no one of them agreed with the other ... the limit...*". On testing, he highlighted the difficulties in waiting for a 28-day result – what should be done with those deliveries of cement? He favoured a 7-day test as obligatory with a minimum tensile strength of 400 lbs/in$^2$ (0.65kN/m$^2$).

**H. Faija** felt that excessive water would simply lead to a porous concrete. On testing samples made with sand, he had difficulty in accepting that sand that was sufficiently uniform could be found. He agreed with Grant that testing by compression was of little value, but disagreed with the possible use of salt-water in the mix, his own results having shown reduced strengths.

**Gustav Grawitz** supported three points:

1) Uniform testing of Portland cement;

2) Establishment of official and independent testing rooms (labs);

3) Valuation of cement according to quality.

On the fineness of grinding, he noted that material coarser than 9 meshes/mm$^2$ would be rejected as worthless in Germany where some cements were prepared to 3-10% greater than 50 mesh / mm$^2$.

He noted Scott & Redgrave's suggested minimum tensile strength of 112 lbs/in$^2$ (0.18kPa), contrasting it with German rules proposing 0.27kPa. Those making higher strength cements were favouring higher requirements, indeed some could reach 0.37kPa with a few up to 0.56kPa.

### Oleson et al (2004) on Roman hydraulic concrete

This paper summarises work under the ROMACONS research project. Investigates the use of volcanic ash from near Puteoli (pulvis puteolanus – later pozzolana) in the production of hydraulic mortar / concrete. Identifies text by Vetruvius from around 25BC giving example mixes of pozzolana sand from the Bay of Naples (2 parts) with lime (1 part) to produce hydraulic mortar.

Notes development of the harbour at Puteoli (near Naples) after 168BC, and at Cosa (north of Rome) suggesting availability of hydraulic mortar / concrete sometime after 200BC. They cite other uses of these materials in the last decades of the 1st century BC. Further discussion on use of timber formwork to hold the nascent concrete, and on construction rates.

They note considerable inherent variability in the mix of lime and pozzolana, and aggregate, and layering in construction, so multiple samples are needed to try to establish composition. Cores taken with a short initial plug – to be reinserted to seal the hole, then a longer core, say 0.5m tall.

Values of density (1550-2160kg/m$^3$, Young's modulus (5000 to 9000, and one at 19000 MPa) and compressive strength (5-8 MPa).

### Discussion and correspondence to Smith (1892), pp98-140, pp141-198

**Harrison Hayter (ICE vice-president)** noted a discussion five years previously ((1875?) in which he had drawn attention to failures of Portland cement concrete. He then rehearsed Bamber's concerns on the lime content of cement, and deleterious influences of high lime content, believing that it should not exceed 60%, and better >55%. He referred to failures at Maryport and at Aberdeen, described by Smith. He had seen a similar failure at a graving dock at Belfast where a weak Portland cement wall was exposed to seawater, washing out the lime content as magnesium hydrate, but which also expanded, lifting and breaking the wall. He contended that the same had occurred at Maryport and Aberdeen.

He noted and approved Bamber's recommendation to use as much water as the mixture would take up, notwithstanding common practice to use as little as possible.

Hayter supported grinding the cement fine, but not excessively, being content with 92.5% passing 2500 meshes per inch. He was suspicious of any cement including magnesia at above 1%.

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[A note on dates. Many of the ICE papers cited in this thesis were read to ICE in the later part of the year. The discussions followed immediately after the paper. The dates of the papers and discussions given in this thesis are therefore the dates of the ICE meeting, and NOT of the later volume of Proceedings which may give a date one year late.]

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# Appendix A – Historical Timeline

| Date | Places and activities | People | Technology |
|---|---|---|---|
| 1626 | Donaghadee first harbour constructed for Viscount Montgomery. | | |
| 1642 | Gordon's map shows pier and basin at Anstruther | | 1642 - Gordon's map |
| 1661 | 1661 - Seaton Sluice pier, Sir Ralph Delavel; 1663 - Construction of Tangier Mole | | |
| 1666 | | 1669 - Sir Hugh Cholmley appointed as Surveyor General at Tangier | |
| 1676 | 1676 - first of Shere's new caissons placed at Tangier | 1676 - Henry Shere appointed at Tangier | 1676 - Use of wooden caissons at Tangier |
| 1682 | | 1682 - Sibbald appointed as Geographer Royal | |
| 1683 | 1683 - Admiralty report recommended abandonment of Tangier. | | |
| 1684 | 1684 - Demolition of the Tangier Mole complete in Feb 1684 | | |
| 1703 | 1703 Cockenzie pier and harbour on Adair's map | | 1703 - Adair's map (of Scotland?) |
| 1709 | | | 1709 - Abraham Darby smelted iron using local ore and coal (coke) |
| 1711 | 1711 - Bull Walls at Dublin started. Great South Wall completed in 1796 (Cox & Gould) | | |
| 1712 | | | 1712 - First Newcomen steam engine installed at a coal mine near Dudley |
| 1716 | 1716 Dublin Great South Wall started | | |
| 1717 | 1717 Dunbar East Pier completed | | |
| 1724 | | 1724 John Smeaton born, Leeds | |
| 1734 | 1734 Scarborough protected by West and East piers | | |
| 1742 | | | 1742 - Roy's map of Scotland |
| 1747 | 1747 Eymouth Old Pier constructed, William Crow | | 1747 - Roy's map (?) (Graham) |
| 1750 | 1750 Ramsgate - Ockenden designed east pier, of stone | | |
| 1752 | 1752 Rothesay Pier, Bute (started); Scarborough Vincent Pier completed | | |
| 1753 | 1753 Anstruther - second pier added (Graham) | | |
| 1755 | | | 1755 - Roy's map (of Scotland?) |
| 1756 | 1756 Blyth harbour North dyke | Smeaton experiments with limestone and pozzolanas | |
| 1757 | | Thomas Telford born | 1757 - New Eddystone lighthouse started, Smeaton, using lime / pozzolanic mortar |
| 1759 | | | 1759 - Carron ironworks established at Falkirk |
| 1761 | | June 1761 - John Rennie born at East Linton | |
| 1766 | | | 1766 - Tent & Mersey canal started, James Brindley / Hugh Henshall |
| 1767 | 1767 Eyemouth North Pier (designed by Smeaton) completed: Poolbeg Lighthouse founded on timber caissons. | | |
| 1768 | | | 1768 - Staffordshire & Worcester Canal started, 46 miles, completed 1772 |
| 1770 | 1770 Smeaton Pier, St Ives constructed by Thomas Richardson. | | |
| 1772 | | 1772 James Brindley died | 1772 - Birmingham Canal Company opened 22.5 mile length to Aldersley, Birmingham to Wolverhampton. |
| 1773 | 1773 Eyemouth North Pier complete, Smeaton | | 1773 Ramsgate - Steam engine fitted to a lighter to dredge sand; |
| 1774 | 1774 Aberdeen North Pier started, Smeaton | | 1774 Ramsgate - Smeaton recommended sluicing basins; Groignard wooden floating caisson with 8 compartments; steam paddle vessel *Palmipede* on the River Doubs |
| 1775 | | | 1775 - Harecastle Tunnel completed on the Trent & Mersey Canal; Boulton & Watt partnership formed; Ainslie's map (of Scotland?) |
| 1780 | 1780 Aberdeen North Pier, Smeaton Phase 1 complete | | |
| 1781 | | 1781 George Stephenson born | 1781 - James Watt patented a rotating steam engine |
| 1784 | | 1784 Rennie visited Boulton & Watt works in Birmingham | |
| 1786 | 1786 Seahouses harbour, Robert Cramond | | |
| 1789 | | | 1789 - Smeaton used Papin's (1689) design of air pump for a diving bell |
| 1792 | Royal Harbour, Ramsgate, formed by two breakwaters of Portland and Whitby stone, filled by rammed chalk, gravel and lime, designed by William Etheridge. | 1792 John Smeaton died | 1792 - Hay Inclined Plane at Coalport to link Shropshire Canal to River Severn |
| 1793 | France declared war | | |
| 1795 | | | 1795 - Boulton & Watt making steam engines at Smethwick |
| 1796 | | | 1796 - Roman cement patended by James Parker |
| 1797 | 1797 Spithead and Nore mutinies | | |
| 1798 | 1798 French force under General Humbert landed in Co. Mayo. | | |
| 1800 | 1800 Charlestown harbour, St Austell completed to design by Smeaton | | 1800 - Henry Maudslay invents screw-cutting lathe, presages standard bolts etc. |
| 1801 | 1801 Union with Ireland | | 1801 - steam powered pile driving and pumping in London Docks, John Rennie |
| 1802 | 1802 Peace with France | | |
| 1803 | 1803 War with France | 1803 Robert Stephenson born | 1803 - Maiden voyage of (steam powered) *Charlotte Dundas* on River Carron; Surrey Iron Railway opened Wandsworth to Croydon |
| 1804 | | | 1804 - Steam loco ran on Welsh colliery tramroad |
| 1805 | Battle of Trafalgar; Cobb, Lyme Regis, modified by William Jessop | | |
| 1806 | | | 1806 - Admiralty commission John Rennie and Joseph Whidbey to plan a breakwater |
| 1807 | Fraserburgh North Pier stated, Rennie / Stevenson | | |
| 1808 | Folkestone Piers by Telford | | 1808 - First seagoing voyage by paddle steamer *Albany* from Hudson River to Delaware River |
| 1809 | Ardrossan South Pier completed | | |
| 1810 | Aberdeen North Pier, Telford extension started | Howth Pier supervised by John Aird under John Rennie | 1810 - Jolliffe & Banks build steam dredger for Ramsgate |

| | | | |
|---|---|---|---|
| 1811 | Scarborough East Pier completed | | 1808 to 1812 - steam pump used for Corpach sea lock construction |
| 1812 | Aberdeen South Pier, Telford / Gibb started. Plymouth breakwater started | Jonathan Pickernell extended Whitby | 1812 - 9hp Boulton & Paul steam pump used for Clachnaharry sea lock construction; John Rennie used a diving bell to found pierheads at Howth. |
| 1813 | Rennie used diving bell at Ramsgate to level foundations and set foundation stones | | |
| 1814 | | | 1814 - steam-powered bucket dredger used at Dublin |
| 1815 | Feb 1815 - Napoleon escaped from Elba; June 1815 - defeated at Battle of Waterloo | | |
| 1817 | Fortrose Harbour completed, Telfod / John Watson. Dun Laoghaire constructed to design by John Rennie; Cullen North pier by William Minto to design by Telford. | | 1817 - P.S. Tug, first steam tug boat launched by Woods Brother, Glasgow |
| 1818 | Gourdon Harbour started by James Farquar; Fraserburgh South Pier started, Rennie / Stevenson | John Gibb - resident engineers at Banff | |
| 1819 | Dublin North Bull Wall proposed, completed in 1825 (Cox & Gould) | | 1819 - Paddle steamer *Savannah* makes Atlantic crossing |
| 1820 | Portpatrick north breakwater started, Rennie; | 1820 - Thomas Telford, President ICE (1820 to 1834) | 1820 - Royal Navy steam vessel *Monkey* built |
| 1821 | Donaghadee new harbour designed by John Rennie (snr) and supervised by John Rennie (jnr). Napoleon dies on St Helena | John Rennie died; | |
| 1823 | Admiralty give Dover harbour to DHB; Wick Old Pier built (started?); Dunmore East harbour constructed by Alexander Nimmo. | | |
| 1824 | Whitehaven West Pier started, Joseph Whidbey / Sir John Rennie | | 1824 - Joseph Aspdin patented Portland Cement |
| 1825 | Stonehaven, Robert Stevenson | | 1825 - Paddle steamers *Dasher* and *Arrow* used on the Portpatrick - Donaghadee mail route. Stockdon and Darlington railway opens |
| 1826 | 1826 Crail West Pier designed by Robert Stevenson & Sons (Graham) | 1826 Crail West Pier constructed by John Gosmand and Alexander Wishart | |
| 1827 | | | 1827 - Aberdeen's first steam tug |
| 1828 | 1828 Banff Harbour extension completed, John Gibb: Crail West Pier completed | 1828 - Bindon Blood Stoney born | |
| 1829 | | | 1829 - 'Rocket' wins the Rainhill trial; CA and J Deane designed first air-pumped diving helmet |
| 1830 | George IV dies, William IV accedes; Liverpool and Manchester railway opens. Whitehaven West Pier (complete); | | |
| 1831 | Wick new harbour completed, James Bremner; Seaham inner harbour, William Chapman: two piers at Cove harbour | | |
| 1833 | Cockenzie East and West Breakwaters rebuilt to design by Robert Stevenson & Sons | Cockenzie rebuilt by Caddell | |
| 1834 | | Thomas Telford died; James Walker President ICE (1835-1845) | |
| 1835 | Reay harbour completed, James Bremner | | |
| 1836 | | | 1836 - Stothert & Pitt manufacture steam locomotives |
| 1837 | William IV dies, Victoria accedes. Branderburgh Harbour, James Bremner / Alexander Gibb; damage to Banff repaired by James Gibb | | 1837 - Production of diving suits and helmets by Siebe, later Siebe Gorman; Grand Junction Railway opened (Warrington to Birmingham) |
| 1838 | | | 1838 - London to Birmingham railway reached Birmingham; Wm Armstrong designed hydraulic powered crane; *Sirius* steam packet crossed Atlantic under sustained steam power |
| 1839 | Portpatrick north pier damaged | | Ramsgate - slipway capstans still manual, 45 men heave vessel of 400t; |
| 1840 | Dover - Royal Commission favour deep water harbour, cost at £2,000,000. | | |
| 1841 | Arbroath Harbour started by James Leslie | | 1841 - Hill patent for steam vehicle transmission (the differential) |
| 1842 | | | 1842 - Birmingham and Liverpool Canal trialed steam tugs, judged uneconomic and reverted to horses; Clayton, Shuttleworth & Co established and start production of portable steam engines. |
| 1843 | Hynish (complete); Kilrush (complete). Brixham to desgn by James Rendel. | | 1843-45 - Thomas Stevenson's wave force measurements at Skerryvore |
| 1844 | | (Sir) William Mathews born, Penzanze | 1844 - General Steam Carriage Co formed to work Hill's patents: Bertha - steam powered drag dredger built at Bridgewater - see http://www.worldofboats.org/boats/view/bertha/21 |
| 1845 | | Sir John Rennie, president ICE (Jan 1845 to Jan 1848) | 1845 - *HMS Rattler* vs *HMS Alecto* trial demonstrates superiority of screw propellor vs paddles |
| 1847 | Hartlepool North Pier started; Holyhead and Alderney new breakwaters started, Rendel / Hawkshaw. Admiralty Pier, Dover, extended. | | 1847 - Armstrong set up Elswick works to manufacture cranes |
| 1848 | St Catherine's breakwater started | 1848 - Joshua Field ICE President (1848-1850) | |
| 1849 | Portland south breakwaters designed by James Rendel as broken slope rubble mounds. | | |
| 1850 | Fraserburgh Balaclava Pier started, John Gibb | 1850 - Sir William Cubitt President ICE (1850-1852) | |
| 1851 | Great Exhibition | | 1851 - Stothert & Pitt exhibited a hand-powered crane at the 1851 Great Exhibition |
| 1852 | Lybster Harbour improved, Christopher Moses / Joseph Mitchell | James Meadows Rendel President ICE (1852 to 1853) | |
| 1853 | | | |
| 1854 | Blyth outer breakwater started by James Abernethy. | James Simpson President ICE (1854-1856) | |
| 1855 | River Tyne Piers started, Benjamin Lawton to James Walker design; Nether Buckie (complete). Crimea War | 1855 - Philip John Messent appointed RE to construction of North and South Tyne Piers; | 1855 - 10hp steam loco at Muckle Flugga lighthouse; Bessemer patented his steel making process. |
| 1856 | St Catherine's (complete). Crimea War | 1856 - Stoney appointed Assistant Engineer to Dublin Port Authority; Robert Stevenson President ICE (1856 to 1857) | 1856 - Clayton, Shuttleworth & Co had produced 2200 portable steam engines since 1842 |
| 1857 | Muckle Flugga Lighthouse completed, D&T Stevenson / Alan Brebner | Joseph Lock President ICE (1857-1859) | |
| 1858 | Indian Mutiny | | |

| | | | |
|---|---|---|---|
| 1859 | | George Bidder President ICE (1859-1862) | 1859 - Steam tugs in construction at Alderney; *Great Eastern* entered service |
| 1860 | Prince Albert dies | | 1860 - *HMS Warrior* launched |
| 1861 | | Francis Pickernell reconstructing Whitby. | 1861 Stothert & Pitt exhibited a 6t travelling crane at Paris (steam-poweed?) |
| 1862 | River Tyne Piers contract taken in-house | Stoney appointed Chief Engineer to Dublin Port Authority; Sir John Hawkshaw President ICE (1862 - 1863) | 1862 Philip John Messent making 40t concrete blocks for Tyne Piers using his own design of concrete mixer; Stothert & Pitt show 6t travelling crane at Paris |
| 1863 | Wick south breakwater, D&T Stevenson; River Tees North and South Gare breakwaters started | | 1863 Goliath crane lifting 50t blocks |
| 1864 | | 1864 - John Robinson McClean President ICE (1864-1866) | 1864, Steam dredgers on River Clyde, paper by Duncan to IMechE |
| 1865 | | | 1865 (approx) - steam locomotives being used by Thomas Stevenson at Wick |
| 1866 | Dover Admiralty Pier started; Anstruther east and west breakwaters, D&T Stevenson | 1866 - Sir John Fowler Predicent ICE (1866-1868) | |
| 1868 | | 1868 - Charles Hutton Gregory President ICE (1868 - 1870) | |
| 1869 | Aberdeen South Breakwater started, WD Cay. Suez canal opened. | | |
| 1870 | Haldon Pier, Torquay, using concrete blocks (4.5$m^3$), placed by divers | 1870 - Charles Blacker Vignoles President ICE (1870-1872) | |
| 1871 | Dover Admiralty Pier complete | | |
| 1872 | | Thomas Hawksley - President (1872-1873) | Dec 1872 - failure of 1350t monolith at Wick; two tank locos from construction of Alderney breakwater in plant sale |
| 1873 | Portpatrick north and south breakwaters abandoned | | 1873 (or earlier) - helmet divers (up to 12 at a time) used at Aberdeen South Breakwater |
| 1874 | Aberdeen North Pier, Dyce Cay extension started; Buckie extended, Dyce Cay / David Cunningham / James Barron | Thomas E. Harrison President ICE (1874-1876) | |
| 1875 | Fraserburgh Balaclava Breakwater, Abernethy and Bostock | | 1875 Concrete bags of 50t being placed by bottom-opening barge at Aberdeen |
| 1876 | Holyhead | George Robert Stephenson President ICE (1876 to 1878) | 1876 - *HMS Inflexible*, first naval vessel powered by vertical compound steam engines |
| 1877 | Fraserburgh Balaclava breakwater (complete); Dover - damage to parapet; Macduff, Stevenson | | |
| 1878 | | John Frederick La Trobe Bateman ICE President (1878-1880) | |
| 1879 | | | |
| 1880 | Newhaven | William Henry Barlow (ICE President 1880-1881) | |
| 1881 | | James Abernethy ICE President (1881-1882) | |
| 1882 | Portsoy East Harbour, James Brand Glasgow) supervised by John Willet | J Watt Sandeman appointed as engineer to Blyth Harbour Commisioners; William Armstrong ICE President (1882-1883) | |
| 1883 | | James Brunlees ICE President (1883-1884) | |
| 1884 | | Joseph Bazalgette ICE President (1884-1885) | 1884 Concrete bags of 104t being placed by bottom-opening barge at Newhaven |
| 1885 | Sunderland outer harbour North Pier, Henry Hay Wake | Frederick Joseph Bramwell ICE President (1885-1886) | 1885 - Stothert & Pitt selling Titan cranes for 50t blocks |
| 1886 | Cullen North Pier, RC Brebner / John Willet; Ardrossan new breakwater, Robert Robertson | Edward Woods ICE President (1886-1887) | |
| 1887 | | George Barclay Bruce ICE President (1887-1889) | |
| 1888 | | | |
| 1889 | | Sir John Coode President ICE (May 1889 to May 1891) | |
| 1891 | Sunderland outer harbour South Pier, Henry Hay Wake | George Barkley ICE President (1891-1892 | 1891 Stothert & Pitt supplied Titan crane weighing 576t for Peterhead, fed with 50t blocks by an auxiliary Goliath |
| 1892 | Ardrossan; Peterhead South Breakwater started, Sir John Coode. | 1892 - Sir John Coode died: Harrison Hayter ICE President (1892-1893) | |
| 1893 | | Alfred Giles ICE President (1893-1894) | |
| 1894 | | Sir Robert Rawlinson ICE President (1894-1895) | |
| 1895 | | Sir Benjamin Baker ICE President (1895-1896) | |
| 1896 | | Sir John Wolfe-Barry ICE President (1896) | 1896 Steam powered barge used at Peterhead. |
| 1897 | | Philip John Messent died, following storm damage to North Tyne Pier | |
| 1898 | German naval expansion. Dover outer harbour - Contract for expansion awarded to S Pearson & Son, contract price £3.3 million | Stoney retired from Chief Engineer to Dublin Port Authority; Sir William Henry Preece ICE President (1898 – 1899) | 1898 - major storm at Peterhead leading to block displacement |
| 1899 | Seahouses harbour extended, Sir John Coode / Watt Sandeman; Seaham harbour extended in concrete blocks by S. Pearson, HH Wake / PW Meik | Sir Douglas Fox ICE President (1899-1900) | 1899 - *HMS Viper* - first steam turbin powered destroyer |
| 1900 | Blyth Harbour North Breakwater extended | James Mansergh ICE President (1900-1901) | 1900 - major storm at Peterhead and block displacement, see Leonard discussion to Buchan (1984) |
| 1901 | Victoria dies, Edward VII accedes. | Charles Hawksley President ICE (Nov 1901 to Nov 1902) | |
| 1902 | Prince of Wales Pier, Dover by Coode, Son & Matthews. | John Clarke Hawkshaw ICE President (1902-1903) | |
| 1903 | | Sir William Henry White ICE President (1903-1904) | |
| 1904 | Anglo-French entente | Sir Guilford Lindsey Molesworth ICE President (1904) | |
| 1905 | Whitby outer breakwaters started, J Watt Sandeman | Sir Alexander Binnie ICE President (1905) | |
| 1906 | | Sir Alexander Kennedy ICE President (1906-1907) | |
| 1907 | | Sir William Mathews, President ICE (1907-1908) | |
| 1908 | Reconstruction of North Tyne Pier completed by Sir John Jackson Ltd supervised by JW Barry / Coode | Sir James Charles Inglis ICE President (1908-1910) | |
| 1909 | Dover - Expansion of harbour completed. Brixham extended. | 1909 - BB Stoney died in Dublin | |
| 1910 | | Sir Alexander Siemens ICE President (1910) | |

# Appendix B – Orphan Breakwater Test Conditions

## APPENDIX B ORPHAN BREAKWATER TEST CONDITIONS

### *Appendix B1 Wave conditions for Test Series 1*

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0t}$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 | A | 1.42 | 4.6 | 4.7 | 0.005 | 7.29 | 7.29 | 0.83 | 0 | - |
| 2 | 1 | A | 1.87 | 4.9 | 4.9 | -0.029 | 7.29 | 7.32 | 0.83 | 0 | - |
| 3 | 1 | A | 2.3 | 5.4 | 5.4 | -0.162 | 7.29 | 7.45 | 0.79 | 0 | - |
| 4 | 1 | A | 3.84 | 6.0 | 6.2 | -0.238 | 7.29 | 7.53 | 0.73 | 0 | - |
| 5 | 1 | A | 3.79 | 6.0 | 6.3 | -0.323 | 7.29 | 7.61 | 0.72 | 0 | - |
| 6 | 1 | A | 5.94 | 7.2 | 7.2 | -0.381 | 7.29 | 7.67 | 0.66 | 0 | - |
| 7 | 1 | A | 6.89 | 7.3 | 7.7 | -0.399 | 7.29 | 7.69 | 0.63 | 0 | - |
| 8 | 1 | A | 6.93 | 7.3 | 7.8 | -0.472 | 7.29 | 7.76 | 0.61 | 0 | - |
| 9 | 1 | A | 7.2 | 7.8 | 8.2 | 0.008 | 7.29 | 7.28 | 0.51 | 0 | 27 |
| 10 | 1 | A | 7.3 | 7.8 | 8.3 | -0.007 | 7.29 | 7.3 | 0.2 | 0 | 40 |
| 11 | 1 | A | 7.26 | 8.3 | 8.3 | -0.019 | 7.29 | 7.31 | 0.12 | 0 | 37 |
| 12 | 1 | A | 7.45 | 8.3 | 11 | -0.032 | 7.29 | 7.32 | 0.15 | 0 | 433 |
| 13 | 1 | A | 7.58 | 8.1 | 13 | -0.089 | 7.29 | 7.38 | 0.16 | 0 | 109 |
| 14 | 1 | B | 8.7 | 10.9 | 13 | -0.108 | -1.53 | -1.42 | 0 | 0 | 13 |
| 15 | 1 | B | 0.99 | 4.4 | 4.7 | -0.202 | -1.87 | -1.67 | 0.13 | 0.73 | 4.6 |
| 16 | 1 | B | 2.66 | 6.2 | 8.4 | -0.162 | -1.80 | -1.64 | 0.1 | 0.56 | 47 |
| 17 | 1 | B | 2.1 | 6.2 | 10 | 0.060 | -1.79 | -1.85 | 0.11 | 0.55 | 15 |
| 18 | 1 | B | 3.09 | 7.4 | 13 | -0.077 | -1.84 | -1.76 | 0.09 | 0.46 | 17 |
| 19 | 1 | B | 4.01 | 8.5 | 8.4 | -0.077 | -1.81 | -1.73 | 0.07 | 0.42 | 7.1 |
| 20 | 1 | B | 5.19 | 9.9 | 9.8 | -0.001 | -1.90 | -1.9 | 0.09 | 0.38 | 9.1 |
| 21 | 1 | B | 6.18 | 10.7 | 11 | 0.084 | -1.80 | -1.88 | 0.12 | 0.37 | 10.5 |

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0t}$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 22 | 1 | B | 7.24 | 11.3 | 13 | 0.017 | -1.84 | -1.86 | 0.16 | 0.33 | 12 |
| 23 | 1 | B | 8.14 | 12.6 | 14 | 0.132 | -1.65 | -1.78 | 0.2 | 0.38 | 14 |
| 24 | 1 | B | 7.81 | 11.8 | 14 | 0.114 | -1.50 | -1.61 | 0.19 | 0.39 | 13 |
| 25 | 1 | C | 0.85 | 7.9 | 8.1 | -0.010 | -1.50 | -1.49 | 0.12 | 0.54 | 6.2 |
| 27 | 1 | C | 1.01 | 7.9 | 7.8 | 0.050 | -1.53 | -1.58 | 0.12 | 0.53 | 6.4 |
| 28 | 1 | C | 3.44 | 11.4 | 12 | -0.016 | -1.50 | -1.48 | 0.19 | 0.48 | 8.7 |
| 29 | 1 | C | 2.09 | 11.4 | 11 | -0.105 | -1.48 | -1.38 | 0.21 | 0.56 | 8.5 |
| 30 | 1 | C | 2.93 | 13.5 | 14 | -0.153 | -1.45 | -1.3 | 0.27 | 0.53 | 11 |
| 31 | 1 | C | 3.87 | 15.9 | 17.7 | -0.035 | -1.04 | -1.01 | 0.26 | 0.55 | 15.2 |
| 32 | 1 | C | 4.95 | 18.6 | 20.6 | -0.086 | -1.78 | -1.69 | 0.25 | 0.54 | 18.6 |
| 33 | 1 | C | 5.31 | 19.2 | 23.1 | -0.053 | -1.99 | -1.94 | 0.24 | 0.53 | 29.9 |
| 35 | 1 | B | 2.11 | 6.2 | 6.7 | -0.056 | | | 0.09 | 0.66 | 5.6 |
| 36 | 1 | B | 1.37 | 5.7 | 8.7 | -0.056 | | | 0.09 | 0.68 | 6.1 |

## *Appendix B2 Wave conditions for Test Series 2*

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0}t$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 37 | 2 | A | 1.42 | 4.9 | 4.9 | 0.035 | 7.56 | 7.52 | 0.85 | 0 | 53 |
| 38 | 2 | A | 2.02 | 5.1 | 5.0 | -0.184 | 7.56 | 7.74 | 0.86 | 0 | 46 |
| 40 | 2 | A | 3.13 | 5.8 | 5.9 | 0.020 | 7.56 | 7.54 | 0.76 | 0 | 46 |
| 41 | 2 | A | 4.31 | 7.1 | 6.7 | -0.041 | 7.56 | 7.6 | 0.72 | 0 | 39 |
| 43 | 2 | A | 5.84 | 7.3 | 7.5 | 0.047 | 7.56 | 7.51 | 0.71 | 0.02 | 41 |
| 44 | 2 | A | 6.38 | 8.1 | 8.2 | 0.041 | 7.56 | 7.52 | 0.66 | 0.03 | 45 |
| 45 | 2 | A | 6.64 | 8.7 | 8.6 | 0.099 | 7.56 | 7.46 | 0.65 | 0.03 | 43 |
| 46 | 2 | A | 9.25 | 9.0 | 9.5 | -0.077 | 7.56 | 7.64 | 0.63 | 0.03 | 44 |
| 47 | 2 | A | 7.6 | 10.2 | 10 | -0.019 | 7.56 | 7.58 | 0.9 | 0.05 | 35 |
| 48 | 2 | A | 8.57 | 10.2 | 12 | 0.017 | | | 0.59 | 0.06 | 30 |
| 49 | 2 | A | 9.04 | 11.3 | 13 | 0.017 | 5.18 | 5.16 | 0.33 | 0.11 | 14 |
| 52 | 2 | A | 8.91 | 11.9 | 16 | 0.056 | 5.18 | 5.12 | 0.24 | 0.29 | 23 |
| 53 | 2 | B | 0.93 | 4.2 | 4.5 | -0.080 | -0.84 | -0.76 | 0.12 | 0.43 | 4.8 |
| 54 | 2 | B | 1.63 | 5.1 | 5.2 | -0.019 | -0.23 | -0.21 | 0.09 | 0.39 | 5.2 |
| 55 | 2 | B | 2.38 | 6.1 | 6.1 | -0.035 | -0.98 | -0.94 | 0.08 | 0.39 | 5.6 |
| 56 | 2 | B | 2.29 | 6.6 | 6.6 | 0.038 | -0.79 | -0.83 | 0.07 | 0.44 | 6.0 |
| 57 | 2 | B | 2.53 | 6.6 | 6.6 | 0.056 | -0.78 | -0.84 | 0.07 | 0.44 | 6.3 |
| 58 | 2 | B | 3.01 | 7.2 | 7.1 | 0.063 | -0.87 | -0.93 | 0.07 | 0.43 | 7.0 |
| 59 | 2 | B | 3.48 | 7.8 | 7.6 | -0.007 | -0.84 | -0.83 | 0.07 | 0.42 | 7.7 |
| 60 | 2 | B | 4.06 | 8.7 | 8.4 | 0.032 | -0.83 | -0.86 | 0.08 | 0.42 | 9.0 |
| 61 | 2 | C | 1.24 | 8.1 | 7.9 | -0.050 | -0.90 | -0.85 | 0.09 | 0.49 | 6.5 |
| 62 | 2 | C | 1.06 | 8.1 | 8.1 | -0.029 | -0.90 | -0.87 | 0.09 | 0.5 | 8.2 |
| 63 | 2 | C | 1.53 | 9.6 | 9.4 | -0.001 | -0.90 | -0.9 | 0.1 | 0.49 | 8.3 |
| 64 | 2 | C | 2.28 | 11.4 | 12 | -0.077 | -0.90 | -0.82 | 0.17 | 0.43 | 9.6 |
| 65 | 2 | C | 2.65 | 12.6 | 13 | 0.002 | -0.90 | -0.9 | 0.22 | 0.43 | 11 |
| 66 | 2 | C | 3.0 | 13.5 | 14 | -0.071 | -0.23 | -0.16 | 0.24 | 0.43 | 13 |
| 67 | 2 | LE | 3 | 10.3 | 10.2 | 0.020 | | | 0.05 | 0.4 | 6.8 |
| 68 | 2 | LE | 3.03 | 10.3 | 10.1 | -0.193 | | | 0.05 | 0.37 | 6.9 |
| 69 | 2 | LE | 3.06 | 10.3 | 10.0 | -0.329 | | | 0.05 | 0.36 | 6.5 |
| 70 | 2 | C | 3.26 | 14.9 | 16.3 | -0.023 | -1.08 | -1.06 | 0.24 | 0.48 | 14.6 |

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0}t$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 71 | 2 | C | 3.98 | 15.9 | 18.9 | -0.013 | -1.02 | -1.01 | 0.24 | 0.48 | 17.9 |
| 72 | 2 | B | 5.22 | 8.9 | 9.0 | 0.029 | -1.02 | -1.05 | 0.09 | 0.38 | 9.1 |
| 73 | 2 | B | 4.56 | 8.9 | 8.8 | -0.019 | -1.02 | -1 | 0.09 | 0.4 | 8.3 |
| 74 | 2 | A | 1.91 | 4.9 | 5.0 | -0.026 | -1.02 | -0.99 | 0.14 | 0.57 | 6.0 |
| 75 | 2 | A | 2.08 | 5.3 | 5.1 | 0.148 | -1.02 | -1.17 | 0.14 | 0.62 | 5.8 |
| 76 | 2 | A | 3.15 | 5.8 | 5.9 | -0.06 | -1.02 | -0.96 | 0.12 | 0.41 | 5.7 |
| 77 | 2 | A | 4.19 | 6.8 | 6.8 | -0.03 | -1.02 | -0.99 | 0.12 | 0.39 | 6.8 |
| 78 | 2 | B | 0.96 | 4.2 | 4.6 | -0.19 | -1.35 | -1.16 | 0.13 | 0.59 | 4.1 |
| 79 | 2 | B | 1.64 | 5.1 | 5.2 | -0.18 | -1.35 | -1.17 | 0.11 | 0.51 | 4.7 |
| 80 | 2 | B | 2.48 | 6.3 | 6.0 | -0.17 | -1.35 | -1.18 | 0.11 | 0.45 | 5.6 |

*Appendix B3 Wave conditions for Test Series 3*

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0}t$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 81 | 3 | A | 2.18 | 5.1 | 5.2 | 2.79 | 2.79 | 2.62 | 0.87 | 0.02 | 102 |
| 82 | 3 | A | 3.28 | 6.0 | 5.9 | 2.79 | 2.79 | 2.76 | 0.78 | 0.02 | 52 |
| 83 | 3 | A | 4.46 | 7.0 | 6.8 | 2.79 | 2.79 | 2.96 | 0.78 | 0.02 | 40 |
| 84 | 3 | A | 5.61 | 7.2 | 7.4 | 2.79 | 2.79 | 2.89 | 0.72 | 0.02 | 19 |
| 85 | 3 | A | 5.44 | 8.1 | 8.0 | 2.79 | 2.79 | 2.66 | 0.72 | 0.03 | 17 |
| 86 | 3 | A | 6.63 | 8.7 | 8.7 | 2.79 | 2.79 | 2.81 | 0.72 | 0.05 | 11 |
| 87 | 3 | A | 7.72 | 8.9 | 9.1 | 2.79 | 2.79 | 2.97 | 0.65 | 0.04 | 15 |
| 88 | 3 | A | 7.70 | 8.9 | 9.0 | 2.79 | 2.79 | 3.00 | 0.65 | 0.04 | 13 |
| 89 | 3 | A | 7.60 | 8.9 | 9.0 | 2.79 | 2.79 | 2.88 | 0.65 | 0.05 | 13 |
| 90 | 3 | A | 5.36 | 9.0 | 9.4 | 2.79 | 2.79 | 2.62 | 0.65 | 0.06 | 13 |
| 91 | 3 | A | 7.74 | 9.0 | 9.4 | 2.79 | 2.79 | 2.81 | 0.61 | 0.06 | 9.1 |
| 92 | 3 | A | 7.85 | 10.0 | 9.8 | 2.79 | 2.79 | 2.96 | 0.61 | 0.07 | 10 |
| 93 | 3 | A | 7.29 | 10.0 | 9.6 | 2.79 | 2.79 | 2.90 | 0.61 | 0.06 | 9.4 |
| 94 | 3 | A | 7.48 | 10.2 | 10 | 2.79 | 2.79 | 2.87 | 0.61 | 0.07 | 11 |
| 95 | 3 | A | 6.76 | 10.5 | 10 | 2.79 | 2.79 | 2.86 | 0.61 | 0.08 | 11 |
| 96 | 3 | A | 7.57 | 9.0 | 9.3 | 2.79 | 2.79 | 2.97 | 0.61 | 0.07 | 8.5 |
| 97 | 3 | A | 8.50 | 10.8 | 12 | 2.79 | 2.79 | 2.80 | 0.54 | 0.11 | 9.7 |
| 98 | 3 | A | 9.16 | 10.7 | 13 | 2.79 | 2.79 | 2.97 | 0.37 | 0.15 | 28 |
| 99 | 3 | A | 8.96 | 10.9 | 11 | 2.79 | 2.79 | 2.58 | 0.26 | 0.22 | 13 |
| 100 | 3 | A | 2.01 | 6.0 | 5.0 | 1.38 | 1.38 | 1.46 | 0.12 | 0.09 | 13 |
| 101 | 3 | B | 1.92 | 6.0 | 5.9 | 1.38 | 1.38 | 1.58 | 0.26 | 0.10 | 9.4 |
| 102 | 3 | C | 2.33 | 11.4 | 11 | 1.38 | 1.38 | 1.28 | 0.26 | 0.22 | 11 |

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0}t$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 103 | 3 | A | 3.03 | 5.8 | 5.8 | 0.33 | 0.33 | 0.35 | 0.10 | 0.18 | 6.2 |
| 104 | 3 | B | 3.10 | 7.2 | 7.1 | 0.33 | 0.33 | 0.54 | 0.10 | 0.27 | 8.1 |
| 105 | 3 | C | 3.24 | 13.5 | 14 | -0.30 | -0.30 | -0.19 | 0.29 | 0.45 | 13 |
| 106 | 3 | A | 3.61 | 6.4 | 6.7 | 0.33 | 0.33 | 0.34 | 0.01 | 0.40 | 6.9 |
| 107 | 3 | B | 4.20 | 8.3 | 8.5 | 0.33 | 0.33 | 0.53 | 0.01 | 0.42 | 8.8 |
| 108 | 3 | C | 3.75 | 15.9 | 18 | -0.75 | -0.75 | -0.75 | 0.01 | 0.60 | 17 |
| 109 | 3 | A | 4.96 | 7.4 | 7.7 | -0.75 | -0.75 | -0.82 | 0.01 | 0.45 | 8.9 |
| 110 | 3 | B | 5.27 | 9.4 | 1.0 | -1.59 | -1.59 | -1.60 | 0.01 | 0.46 | 9.1 |

## *Appendix B4 Wave conditions for Test Series 4*

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0t}$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 113 | 4 | A | 4.27 | 7.0 | 6.7 | 0.02 | 9.84 | 9.82 | 0.76 | 0.01 | 71 |
| 114 | 4 | A | 5.53 | 7.2 | 7.4 | -0.07 | 9.84 | 9.91 | 0.74 | 0.01 | 65 |
| 115 | 4 | A | 6.28 | 8.1 | 8.0 | -0.17 | 9.84 | 10.01 | 0.70 | 0.02 | 59 |
| 116 | 4 | A | 7.14 | 8.7 | 8.7 | 0.09 | 9.84 | 9.75 | 0.68 | 0.02 | 58 |
| 117 | 4 | A | 8.30 | 9.1 | 9.3 | -0.03 | 9.84 | 9.87 | 0.63 | 0.02 | 57 |
| 118 | 4 | A | 7.87 | 9.4 | 9.6 | 0.00 | 9.84 | 9.84 | 0.43 | 0.03 | 50 |
| 119 | 4 | A | 7.71 | 9.4 | 10.0 | -0.16 | 9.84 | 10.00 | 0.13 | 0.03 | 72 |
| 120 | 4 | A | 7.98 | 10.2 | 10.8 | 0.11 | 9.84 | 9.73 | 0.20 | 0.03 | 47 |
| 121 | 4 | A | 9.45 | 11.1 | 14.2 | -0.03 | 9.84 | 9.87 | 0.28 | 0.04 | 28 |
| 122 | 4 | A | 8.75 | 11.1 | 12.8 | 0.08 | 9.84 | 9.76 | 0.29 | 0.19 | 25 |
| 123 | 4 | B | 0.94 | 4.2 | 4.4 | -0.03 | 6.27 | 6.30 | 0.11 | 0.58 | 4.0 |
| 124 | 4 | B | 1.30 | 4.6 | 4.8 | -0.04 | 6.27 | 6.31 | 0.13 | 0.54 | 4.2 |
| 125 | 4 | B | 1.62 | 5.1 | 5.2 | -0.03 | 6.27 | 6.30 | 0.12 | 0.50 | 4.4 |
| 126 | 4 | B | 1.84 | 5.5 | 5.5 | -0.10 | 6.27 | 6.37 | 0.12 | 0.45 | 4.6 |
| 127 | 4 | B | 2.03 | 5.9 | 5.9 | -0.02 | 6.27 | 6.29 | 0.12 | 0.46 | 4.9 |
| 128 | 4 | B | 1.62 | 5.1 | 4.4 | -1.24 | -1.05 | 0.19 | 0.10 | 0.06 | 6.1 |
| 129 | 4 | B | 1.84 | 5.5 | 5.9 | -1.28 | -1.05 | 0.23 | 0.11 | 0.10 | 7.6 |
| 130 | 4 | B | 2.03 | 5.9 | 7.0 | -1.30 | -0.99 | 0.31 | 0.08 | 0.14 | 10.4 |
| 131 | 4 | B | 1.01 | 4.2 | 8.3 | -1.34 | -1.11 | 0.23 | 0.09 | 0.17 | 13.6 |
| 132 | 4 | B | 2.19 | 5.9 | 4.4 | -1.77 | -0.99 | 0.78 | 0.14 | 0.01 | 32 |
| 133 | 4 | B | 3.54 | 7.2 | 5.8 | -1.81 | -0.93 | 0.88 | 0.14 | 0.02 | 34 |
| 134 | 4 | B | 4.87 | 8.3 | 7.0 | -1.83 | -0.93 | 0.90 | 0.10 | 0.04 | 20 |
| 135 | 4 | B | 1.06 | 4.2 | 8.1 | -1.87 | -0.99 | 0.88 | 0.09 | 0.08 | 19 |

*Appendix B5 Wave conditions for Test Series 5*

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0t}$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 136 | 5 | A | 1.33 | 5.2 | 5.2 | -1.54 | 4.38 | 5.92 | 0.79 | 0.00 | - |
| 137 | 5 | A | 3.28 | 7.1 | 6.7 | -1.51 | 4.38 | 5.89 | 0.68 | 0.00 | 85 |
| 138 | 5 | A | 4.26 | 7.3 | 7.3 | -1.44 | 4.38 | 5.82 | 0.67 | 0.03 | 40 |
| 139 | 5 | A | 4.92 | 8.0 | 8.0 | -1.55 | 4.38 | 5.93 | 0.61 | 0.03 | 20 |
| 140 | 5 | A | 5.63 | 8.0 | 8.2 | -1.39 | 4.38 | 5.77 | 0.55 | 0.05 | 11 |
| 141 | 5 | A | 5.80 | 8.4 | 8.3 | -1.36 | 4.38 | 5.74 | 0.46 | 0.05 | 9.2 |
| 142 | 5 | A | 5.59 | 7.8 | 8.3 | -1.33 | 2.7 | 4.03 | 0.13 | 0.08 | 19 |
| 143 | 5 | A | 5.61 | 8.2 | 8.4 | -1.30 | 2.7 | 4.00 | 0.12 | 0.08 | 19 |
| 144 | 5 | A | 5.51 | 7.8 | 8.5 | -1.27 | 2.7 | 3.97 | 0.10 | 0.13 | 25 |
| 145 | 5 | A | 0.29 | 4.1 | 4.4 | -1.45 | -0.81 | 0.64 | 0.16 | 0.10 | 31 |
| 146 | 5 | B | 0.72 | 4.2 | 4.4 | -1.36 | -0.81 | 0.55 | 0.12 | 0.07 | 14 |
| 147 | 5 | C | 0.44 | 5.6 | 5.8 | -1.44 | -0.81 | 0.63 | 0.12 | 0.11 | 25 |
| 148 | 5 | A | 1.56 | 5.0 | 4.9 | -1.54 | -0.81 | 0.73 | 0.11 | 0.05 | 15 |
| 149 | 5 | B | 1.85 | 5.9 | 5.8 | -1.54 | -0.81 | 0.73 | 0.09 | 0.06 | 11.8 |
| 150 | 5 | C | 0.96 | 7.9 | 7.8 | -1.51 | -0.81 | 0.70 | 0.15 | 0.10 | 23 |
| 151 | 5 | A | 2.49 | 5.8 | 5.7 | -1.63 | -0.81 | 0.82 | 0.10 | 0.06 | 15 |
| 152 | 5 | B | 2.86 | 7.2 | 7.0 | -1.45 | -0.81 | 0.64 | 0.10 | 0.13 | 15 |
| 153 | 5 | C | 1.52 | 9.6 | 9.2 | -1.60 | -0.81 | 0.79 | 0.10 | 0.09 | 21 |
| 154 | 5 | A | 3.51 | 6.9 | 6.6 | -1.48 | -0.81 | 0.67 | 0.08 | 0.11 | 13 |
| 155 | 5 | B | 3.80 | 8.7 | 8.2 | -1.63 | -0.81 | 0.82 | 0.10 | 0.16 | 16 |
| 156 | 5 | C | 2.03 | 10.4 | 9.9 | -1.30 | -0.72 | 0.58 | 0.14 | 0.18 | 15 |
| 157 | 5 | A | 4.42 | 7.2 | 7.4 | -1.48 | -0.54 | 0.94 | 0.15 | 0.19 | 17 |
| 158 | 5 | B | 5.03 | 9.4 | 9.8 | -1.33 | -0.87 | 0.46 | 0.12 | 0.26 | 15 |
| 159 | 5 | C | 2.70 | 12.6 | 12 | -1.42 | -1.08 | 0.34 | 0.12 | 0.26 | 25 |
| 160 | 5 | A | 5.13 | 8.0 | 8.3 | -1.57 | -1.2 | 0.37 | 0.16 | 0.25 | 18 |
| 161 | 5 | A | 1.14 | 5.0 | 4.9 | 0.07 | -1.65 | -1.72 | 0.22 | 0.69 | 4.8 |
| 162 | 5 | A | 3.07 | 6.8 | 6.7 | 0.01 | -1.83 | -1.84 | 0.16 | 0.49 | 6.9 |
| 163 | 5 | A | 5.02 | 7.8 | 8.6 | 0.10 | -1.86 | -1.96 | 0.16 | 0.47 | 11.4 |
| 164 | 5 | B | 0.70 | 4.3 | 4.5 | -0.05 | -1.44 | -1.39 | 0.21 | 0.90 | 4.5 |
| 165 | 5 | B | 2.47 | 7.2 | 7.0 | 0.10 | -1.77 | -1.87 | 0.16 | 0.64 | 6.4 |
| 166 | 5 | B | 4.35 | 9.4 | 9.8 | -0.02 | -1.95 | -1.93 | 0.12 | 0.52 | 9.7 |
| 167 | 5 | C | 0.95 | 7.9 | 7.8 | 0.07 | -3.18 | -3.25 | 0.18 | 0.67 | 6.6 |
| 168 | 5 | C | 2.06 | 11.1 | 11 | 0.01 | -3.45 | -3.46 | 0.16 | 0.58 | 8.0 |
| 169 | 5 | C | 3.60 | 13.5 | 15 | 0.01 | -3.6 | -3.61 | 0.13 | 0.47 | 14 |

*Appendix B6 Wave conditions for Test Series 6*

| Run No. | Sect. | Series | $H_s$ (m) | $T_p$ (s) | $T_{m,-1,0}$ (s) | SWL (m) | Crest (m) | $R_c$ (-) | $C_r$ (-) | $C_t$ (-) | $T_{m,-1,0}t$ (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 170 | 6 | A | 1.79 | 5.2 | 5.4 | -1.51 | 4.38 | 5.89 | 0.80 | 0.02 | 159 |
| 171 | 6 | A | 4.00 | 6.6 | 7.0 | -1.42 | 4.38 | 5.80 | 0.69 | 0.02 | 158 |
| 172 | 6 | A | 5.69 | 8.3 | 8.8 | -1.48 | 4.38 | 5.86 | 0.59 | 0.05 | 19 |
| 173 | 6 | A | 5.46 | 7.9 | 8.8 | -1.48 | 4.38 | 5.86 | 0.58 | 0.05 | 28 |
| 174 | 6 | A | 5.69 | 7.9 | 8.9 | -1.39 | 4.38 | 5.77 | 0.55 | 0.06 | 24 |
| 175 | 6 | A | 5.65 | 7.9 | 8.8 | -1.45 | 4.38 | 5.83 | 0.52 | 0.06 | 21 |
| 176 | 6 | A | 5.64 | 8.0 | 9.0 | -1.48 | 1.71 | 3.19 | 0.33 | 0.09 | 18 |
| 177 | 6 | A | 5.44 | 8.1 | 10 | -1.51 | 1.71 | 3.22 | 0.22 | 0.25 | 31 |
| 178 | 6 | A | 0.38 | 3.9 | 4.2 | -1.54 | 1.71 | 3.25 | 0.23 | 0.13 | 25 |
| 179 | 6 | A | 1.83 | 5.3 | 5.4 | -1.57 | -2.15 | -0.58 | 0.18 | 0.13 | 28 |
| 180 | 6 | A | 2.81 | 6.2 | 6.6 | -1.42 | -2.28 | -0.86 | 0.18 | 0.21 | 16 |
| 181 | 6 | A | 4.00 | 6.7 | 7.3 | -1.45 | -2.42 | -0.97 | 0.17 | 0.23 | 17 |
| 182 | 6 | B | 0.93 | 4.1 | 4.5 | -1.57 | -2.43 | -0.86 | 0.22 | 0.13 | 17 |
| 183 | 6 | B | 2.24 | 5.9 | 6.3 | -1.42 | -2.3 | -0.88 | 0.17 | 0.20 | 16 |
| 184 | 6 | B | 3.15 | 7.3 | 7.6 | -1.39 | -2.17 | -0.78 | 0.17 | 0.26 | 17 |
| 185 | 6 | B | 4.00 | 8.4 | 9.8 | -1.42 | -2.49 | -1.07 | 0.23 | 0.31 | 19 |
| 186 | 6 | C | 0.97 | 8.0 | 7.8 | -1.39 | -2.48 | -1.09 | 0.24 | 0.30 | 14 |
| 187 | 6 | C | 2.13 | 11.1 | 11 | -1.45 | -2.2 | -0.75 | 0.17 | 0.30 | 18 |
| 188 | 6 | C | 3.08 | 13.7 | 15 | -1.51 | -2.46 | -0.95 | 0.14 | 0.38 | 22 |
| 189 | 6 | A | 4.29 | 16.1 | 18 | -1.57 | -2.14 | -0.57 | 0.11 | 0.40 | 24 |
| 190 | 6 | A | 1.60 | 5.3 | 5.1 | 0.10 | -2.04 | -2.14 | 0.18 | 0.64 | 5.2 |
| 191 | 6 | A | 3.70 | 6.7 | 7.1 | 0.19 | -2.48 | -2.67 | 0.14 | 0.46 | 8.3 |
| 192 | 6 | A | 5.42 | 7.8 | 9.5 | 0.10 | -2.73 | -2.83 | 0.16 | 0.43 | 13 |
| 193 | 6 | B | 0.84 | 4.2 | 4.6 | 0.07 | -2.63 | -2.70 | 0.19 | 0.78 | 5.7 |
| 194 | 6 | B | 3.06 | 7.3 | 7.4 | 0.04 | -2.68 | -2.72 | 0.15 | 0.53 | 7.8 |
| 195 | 6 | B | 5.36 | 9.4 | 11 | 0.10 | -2.8 | -2.90 | 0.12 | 0.42 | 13 |
| 196 | 6 | C | 1.03 | 8.0 | 7.7 | 0.04 | -2.9 | -2.94 | 0.16 | 0.78 | 6.1 |
| 197 | 6 | C | 2.18 | 11.4 | 11 | 0.07 | -2.93 | -3.00 | 0.11 | 0.64 | 8.8 |
| 198 | 6 | C | 3.29 | 13.7 | 15 | 0.04 | -2.81 | -2.85 | 0.10 | 0.56 | 14 |
| 199 | 6 | C | 1.81 | 11.4 | 12 | -1.48 | -2.85 | -1.37 | 0.10 | 0.38 | 13 |
| 200 | 6 | A | 3.91 | 6.8 | 7.6 | -1.45 | -2.78 | -1.33 | 0.14 | 0.29 | 13 |

# Appendix C – Published Papers

APPENDIX C PUBLISHED PAPERS


Allsop W (2017) History of Alderney and Jersey "harbours of refuge" –why did they fail? Proc. Conf. Coasts, Marine Structures and Breakwaters 2017, Liverpool, September 2017, editor Kevin Burgess, pp 3-13, ICE Publishing, ISBN 978-0-7277-6317-4

Allsop N.W.H. (2020) English Channel "harbours of refuge" – a discussion on their origins and 'failures', Proc. ICE, Engineering History and Heritage, https://doi.org/10.1680/jenhh.19.00034, ICE Publishing.

Allsop N.W.H. & Bruce T (2020) Failure analysis of historic breakwater, Part 1 Wick, Scotland, Proc. ICE Forensic Engineering, https://doi.org/10.1680/jfoen.20.00002, ICE Publishing.

Allsop N.W.H. & Bruce T (2020) Failure analysis of historic breakwaters, Part 2 Alderney, Guernsey, and Dover, UK, Proc. ICE Forensic Engineering, https://doi.org/10.1680/jfoen.20.00004, ICE Publishing.

Allsop W, Pearson A. & Bruce T. (2017) Orphan breakwaters – what protection is given when they collapse? Proc. ICE Conf. Coastal Structures and Breakwaters, Liverpool, September 2017, editor Kevin Burgess, pp 697-708, ICE Publishing, ISBN 978-0-7277-6317-4

Allsop W, Pearson A. & Bruce T. (2018) Orphan breakwaters – what protection is given when they collapse? Proc. ICE Maritime Engineering, https://doi.org/10.1680/jmaen.2018.13, ICE Publishing.

Pearson A.N. & Allsop W. (2017) Exploring structural stability of old blockwork breakwaters, Proc. Conf. Coastal Structures and Breakwaters 2017, Liverpool, September 2017, Editor Kevin Burgess, ICE publishing, ISBN 978-0-7277-6317-4

# Appendix D – Geographic Review

# 1 Outline

The summaries in this Geographic review are presented by general geographic area: Ireland; Wales; Scotland; and England. They are intended to identify breakwaters and a few other related structures and give a few summary details. Those breakwaters where individual references give more detail are discussed in more detail in Chapter 12).

# 2 Ireland - Cox R C & Gould M H (1998)

**The Bull Walls, Dublin (HEW 3016)**

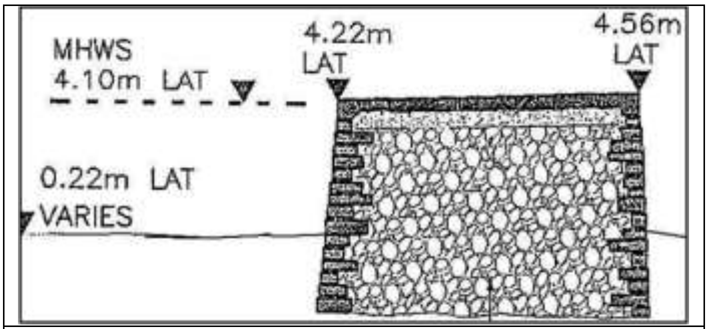


**Figure D.1 Section through Dublin Great South Wall**

Protecting the Dublin harbour channel from siltation (and waves), the Bull Walls started in 1711 with a wall (termed jetty) from Ringsend to Pigeon House Fort, then on to the eastern spit of South Bull, a total distance of 5.4km. The jetty was formed by timber caissons built at Ringsend and floated downstream, then being sunk into position by filling with rubble. It was completed in 1730. The first section was replaced by a pair of low-lying stone walls with sand or stone infill, Figure D.1. The Great South or Bull Wall was completed in 1796, including the forming of Pigeon House Harbour. The North Bull Wall was proposed in 1819 and completed in 1825.

**Poolbeg Lighthouse (HEW 3051)**

The lighthouse was formed as a masonry tower at the end of the Great South Wall founded on timber caissons completed in 1767. The tower was extended in height in 1813, and its toe was protected by large concrete blocks in the 1870s, see Stoney (1874).

**Howth Harbour (HEW 3055)**

East pier was started in 1807 to a design by Cpt. George Taylor, but the pier end collapsed in 1809 and Taylor resigned. Later supervised by John Aird under direction of John Rennie. Stone was from Kilrock, but later used granite from Dalkey. The West pier was added 1810-1813 to complete the harbour, but mail packets only transferred in 1818, and then away to Dun Laoghaire in 1826. Rennie used a diving bell in 1812 to help found the pierheads. Note use of Runcorn (sand)stone, easy to cut, but hardens when submerged in (sea?)-water. Piers were battered, tapering from 61m at the base to 26m at HW, with a 12.5m wide roadway.

**Dun Laoghaire (HEW 3014)**

Constructed between 1817 and 1827 to a design by John Rennie. The bases of both breakwaters were formed by Runcorn stone (sandstone) with upper layers protected by granite from Dalkey transported by (funicular?) railway. Granite rubble core between the walls was loose tipped.

**Courtown Harbour (HEW3037)**

Two shore-normal piers were completed 1834-47 under direction of Francis Giles. Sluices and an overflow were included to assist scouring the entrance channel. South pier was rebuilt and lengthened in 1871.

**Rosslare Harbour (HEW 3092)**

Mainly open jetties, then protected in 1904-6 by mass concrete quays and seawall, but few traces of earlier works remain as the harbour was rebuilt and enlarged in the late 1960s and 1990s.

**Donaghadee Harbour (HEW 2110)**

The first harbour was built by / for Viscount Montgomery in 1626, then re-built 1775-1785, but not deemed successful. New harbour designed by John Rennie (senior) and supervised 1821-1836 by John Rennie (junior). Breakwater inner faces in ashlar, but outer faces armoured by rubble pitching at 1:5. Mail packets used the harbour 1825 to 1849, and 1865 to 1867 when the mail service moved permanently to Larne.

**Bangor breakwater (HEW 2095)**

Early North piers constructed in 1757, then 1895, but subsumed into the substantial marina expansion using rubble mound breakwaters armoured by 2t SHED units.

**Carnlough Harbour (HEW 1631)**

Pier (~60m long) built around 1800 by Philip Gibbons using dry stone walling to export limestone to Scotland for iron smelting. The harbour was then up-graded from 1853 in creating a new basin, but rapid siltation was induced, in part by orientation of the Gibbons pier trapping longshore drift. Processed lime was exported from 1855.

**Dunmore East Harbour (HEW 3072)**

Original pier (240m long) constructed under the direction of Alexander Nimmo 1823-1825 to serve mail packet steamers from Milford Haven. Vertical on lee-side and using sloped pitching on the seaward face, partly rebuilt under direction of Barry Duncan Gibbons in 1832. A later storm wall (~200m long) was added on top in 1960's with a substantial concrete recurve.

# 3 Wales - Cragg (1997)

**Holyhead Harbour (HEW 1095, breakwater HEW 1099)**

Admiralty Pier designed by John Rennie, completed in 1821, supplemented by Telford's Dry Dock of 1825, and South pier, 1831. New Harbour started in 1847 protected by breakwater engineered by JM Rendel, then by Sir John Hawkshaw and Harrison Hayter.

**Penmaenmawr Sea Viaduct (HEW 829)**

Following failure of his railway seawall in 1846, Robert Stevenson designed an open viaduct along the shoreline with pitched stone slope beneath. Started in 1847, completed in 1848.

**Pothmadog Cob (HEW 1129)**

Rockfill embankment used to reclaim 7000 acres along the Afon Glaslyn, built by John Williams 1808 - 1811, now carries A487 road and Ffestiniog Railway.

**Whiteford Point Lighthouse (HEW 1256)**

Cast iron lighthouse built in 1865, 61ft high, eight courses of cast iron plates, internal and external joining flanges, later strengthened by wrought iron bands.

# 4 Harbours in Eastern Scotland – Graham (1969)

This extended paper records and discusses (the archaeology of) 125 old harbours around the Firth of Tay, Fife coast, and Firth of Forth and coast down to Burnmouth, researched during 1966-68. Most harbours were for fishing, but some exported coal, salt, limestone, fish and grain. To illustrate the importance of seamanship for the region, Graham notes that 'navigation' was taught in Torryburn village school as early as 1793. Sites first mentioned during the 1700s were often connected to the 'new' industries using coal, lime, iron, so: Charlestown, Starley Burn, Carron, Cramond, Kincardine, Kenetpans and Cambus, Portobello and Grangemouth. Later works included Cove, Burnmouth, St Abbs, Fisherrow, Skateraw, and ferries at Newhaven and Queensferry, and on the Tay. He lists breakwaters at: East pier of Dunbar, cellardyke, Crail, Dysart, Pettycur, Pittenweem and St Andrews, North Berwick (abbreviated by storm damage), West Wemyss. Detached breakwaters may have been used to provide wave reduction at Brucehaven, Torry, St Monance. Graham sometimes uses the term 'pier' as "*smaller and less massively built than breakwaters*". Generic sources identified by Graham include Adair (1703) and Roy's map of Scotland (1747-1755).

**Aberdour, Fife**, had a port in 1565, with a pier existing in 1703, and perhaps modified / extended before 1750. The section of the pier is variable, generally 10m wide. Blockwork is variable, and may show evidence of later repairs.

**Figure D.1 Anstruther main (East) breakwater, July 2013**

**Anstruther Easter** existed as a harbour in the 1580s and Gordon's map of 1642 shows a pier and basin, and stone pier some 540ft long existed in 1703, probably subsumed into the central pier. A second pier was added in 1753. Figure D.1 shows the East breakwater in 2013.

**Anstruther Wester** probably had a breakwater by 1604, damaged and repaired in 1620, and perhaps damaged again in "*the great storm of 1655*".

**Balmerino in Fife** was intended to export lime, but by 1760 cargo included wheat and barley, and by 1838 potatoes. An L shaped pier protected a small tidal pocket, probably equipped for tidal sluicing.

**Balmerino Kirkton**, also in Fife, had a pier / jetty of Whinstone blocks with rubble infill, probably supporting salmon exports.

**Belhaven harbour, East Lothian**, was certainly established before 1153, and was designated a 'free port' by 1370, and probably lasted to 1814. By 1966 the harbour area had been reclaimed.

**Blackness, West Lothian,** was constructed from 1465 using stone and lime from the castle whose demolition had been ordered. A pier shown in 1755 was probably in ruins by 1843, reduced to an "*ill-defined rickle of stones, all below high-water*".[1] By 1895, it had been restored using rough irregular masonry with cemented crest, but again partly ruined before 1966. The muddy bay to the south of Blackness Castle contains "*a long rickle of debris … presumably once a breakwater protecting the N. part of the bay*". The foundations of both faces were formed by rough broken whinstone blocks with smaller core, and suggest it was about 3m wide.

**Braefoot Bay** had a jetty formed of large blocks and smaller fill about 4-5m wide, possible later recycled in other structures.

The pier at **Bridgeness** appears on Ainslie's map of 1775, was lengthened before 1843 and altered since 1856 when it consisted of a single pier. A second pier was later added. The face of the original pier is formed by large drystone blocks, dressed and coursed. The crest uses large slabs with occasional iron cramps.

**Brucehaven harbour** is shown on Roy's 1755 map, approximately 130m long and 8m wide. The east face is formed by squared, coursed blocks, see Graham's photo in Figure D.2. At some stage the entrance was protected by a detached breakwater of large blocks, now reduced to the lowest courses.

**Figure D.2 Brucehaven Pier**

*From Graham (1968)*

**Burnmouth (previously Cramesmouth)** may have existed in 1100, but no harbour is shown explicitly on maps earlier than 1830. The harbour was enlarged in 1879. The pier is formed by rusticated ashlar, battered on both sides.

[1] Definition of a rickle: a small stack or pile, generally of stones.

**Burntisland harbour** was originally simply protected by the island. Most early structures have now been built over, so little remains of archaeological relevance. The remains of two piers to the east (Bath House) suggest piers / jetties for shipping lime in early 1800s.

**Carronshore** was described in 1723 as a good harbour, although Roy's map of 1747-55 shows no harbour works. There is however evidence for a harbour from 1757 and a plan of 1797 shows mooring poles, a Stone Pier, and a Wharf. The only structure remaining is formed by massive stone blocks secured by iron cramps.

**Cellardyke, Fife,** is mentioned in 1579, when authorisation for repair was given. By 1623 it was regarded as decayed and a futher repair grant was obtained in 1625. In 1703 the harbour basin was "*pretty deep and covered by a hewen Head of Stone*" presumably a blockwork breakwater wall. The pier / breakwater shows in Roy's map (1747-55). Before improvements started in 1829, the breakwater appears to have doubled its length, added a cross-pier and a quay. Those improvements, and further in 1853 improved the basin and quay, and changed the cross-pier. The inner part of the breakwater, presumably the oldest, used large square blocks set vertically. The (newer?) outer part uses "*thin blocks, neatly dressed and well coursed*". The West pier also uses neatly coursed, but thicker blocks.

The inner harbour basin at **Charlestown** was protected by a pier formed by large blocks at one end and (smaller?) coursed blocks.

**Figure D.3 Cockenzie East Breakwater**
*Parapet wall showing iron strap, May 2013*

**Cockenzie, East Lothian**, is recorded as a free haven in 1592. Adair records "*a harbour within a stone pier*" in 1703. By 1722 the harbour had been enlarged and connected to the Tranent colliery railway. Colliery and harbour were sold in 1774, and the harbour was rebuilt by Caddell by 1833 to a design by Robert Stevenson & Sons. The east Breakwater is built of squared red-sandstone blocks in courses, with some "*rusticated works in parts of the seaward face*".[2] The parapet of 8.5ft high by 2ft thick (2.6m x 0.6m) is "*secured along the top by a stout iron strap*", see Figure D.3.

[2] Rusticated: rough-surfaced masonry blocks having bevelled or rebated edges producing pronounced joints.

The West breakwater (Figure D.4) "*has rusticated masonry*" in its outer portion, with " *a patch of vertically set slabs further in, no doubt representing a repair*".[3]

**Figure D.4 Cockenzie West Breakwater, May 2013**

The two piers at **Cove in Berwickshire** were finished in 1831, both built of "*red, grey and yellowish sandstone blocks, very well cut and coursed, with iron strapping ... along the edges of upper surfaces and on external angles".* The block bedding was cut into the underlying rock down to the bottom of LW. The South Pier is paved by large slabs, partly replaced by concrete.

**Figure D.5 Crail harbour**

*From Graham (1968)*

**Crail** may have had various harbour structures since perhaps 1512, or perhaps 1537. Certainly there were craft moored in a harbour in 1553, and a substantial breakwater is indicated in the late 1500s, although much damaged in 1587 by *"... ventis orientabilis devastati*", and again in 1655. The pier was described in 1689 as of "*large stone work built far within the sea*". Work done in 1745 may have involved alterations as well as repairs, still in progress in 1760. The pier face is formed by "*large rough blocks, partly dry-built and secured by wooden wedges, and partly pointed ... on a heavily battered and mortared base*" (Figure D.5).

[3] The confidence of the "no doubt" may be a little surprising as the section with vertical blocks shown in Figure 12.5 might perhaps be earlier!

The (minor) West Pier at Crail was designed by Robert Stevenson & Sons around 1820s and constructed by John Gosman (and Alexander Wishart). Both faces "*show vertically set masonry, partly mortared and neater than that of the main pier*".

**Crombie point, Fife,** features a long pier (280m) running approximately SW across inter-tidal mud / rocks, then S and finally SE. It is mainly formed of large squared blocks in courses, with undressed material in the core, and the crest unpaved (or it was so around 1966). The outer end partly encloses a [dredged or excavated?] tidal pocket.

The harbour at **Culross, Fife,** was formed by two piers, the Long Pier and the New Pier (possibly from 1689). The Long Pier was detached, joined to the shore-linked New Pier by a timber trestle or 'platform'. The Long Pier appears to have been formed by large roughly squared blocks set in courses, and the crest was paved by large setts. Some parts, perhaps repaired, use split un-squared stones set vertically (attributed to early 1900s).

The New Pier at **Culross** contains work of two separate phases, both using very large dry-jointed stone blocks, laid in courses. Blocks in the lower part "are distinguished by stugging or horizontal droving", whilst those in "the upper are simply hammer-dressed". The top of the lower part was paved by large setts.

**Donibristle, Fife**, has had two harbours, New and Old. The jetty forming the Old harbour dates from before 1768, perhaps a little after 1720, and was formed by "*large dry-stone blocks, roughly dressed and poorly coursed*". The crest "*is roughly paved with smallish unshaped blocks*". The New Pier, perhaps less exposed to wave action as it is more shoreward, is formed by more regularly dressed blocks, well coursed. The toe is protected by an apron of smaller blocks, not bonded to the pier.

**Dunbar (or Lamerhaven) in East Lothian** is protected from wave action by the East Pier, perhaps from 1574. About 280m long, the East Pier is formed of "*many varieties of masonry including beach-boulders, large roughly-dressed blocks, and slabs set obliquely or vertically*." It was damaged (and later repaired) in notable storms of 1655 and 1906. The minor pier probably dates from 1717 formed by squared and well-course blocks. The harbour entrance has changed a number of times, usually occasioned by different attempts to use wave-excluding booms (stop-logs) in storms.

**Dysart, Fife** has had a harbour (or two) since before 1534. The Old harbour was protected by the Jetty, formed by cutting out from a natural rock outcrop. The West Harbour (now abandoned) was protected by 1703 by the "strong bulwark of stone" of the East Pier. Both East and West piers are shown in maps of 1750, but bad damage is recorded in storms of 1705, 1724, 1803, 1837-8 and 1843. It is suggested that some materials in the West Harbour piers were removed from the Old harbour. The remaining structures are formed by rusticated masonry of early 1800s, and/or by vertically set blocks, perhaps the earlier work.

**Eyemouth harbour** has collected anchorage dues from 1288, and had riverside quays on the Eye Water by 1703. In 1768 the North Pier on the right bank was constructed by Cramond of Dunbar to a design by Smeaton, and another on the left bank before 1750. Stone used was a local conglomerate in large blocks, but few parts of the construction remain original.

The East Pier at **Fisherrow, Midlothian**, is mainly formed by large red-sandstone blocks, roughly course and in some places rusticated, topped by a stout parapet. The West Pier was constructed in 1843-4 by Kinghorn of Leith to the design of Robert Stevenson and Sons. The red-sandstone masonry is more neatly dressed than on the East Pier, placed at a marked batter in regular courses.

**Kinghorn, Fife**, was protected by a new pier started around 1698, and "*lately built*" by 1710, but ruined by 1846. Remnants survive showing "*several courses of very large blocks*".

**Kikcaldy East Pier** a "*long, high and strong Mole of hewn Stone*" probably dates from 1648, certainly before 1703. It collapsed in 1723. A West pier was added around 1754 but later removed to enlarge the harbour basin.

Originally a river port, **Leith** had acquired a bulwark to contain the river by 1508, showing as a pier by 1560. The main problems appear to have been siltation, managed by a "wooden Fence" and then by Stonern Pier about 1722. By 1777 a timber pier was reinforced by boulders.

**Newhaven, Edinburgh**, was chosen as the site for a naval dockyard by James IV before 1506. By 1556 there were two piers curving out from the shore to a narrower entrance, but no explicit trace is shown of the dockyard. Some remains of the harbour survived to 1703, and then 1760.

The ferry jetty at **Newport, Fife**, designed by Telford was constructed in 1823-6. It may still exist, but was inaccessible to Graham in 1966.

Some form of harbour at **North Berwick** may have existed well before 1658, although earlier works have been subsumed into more recent construction. The breakwater is formed of boulders and large rough blocks, irregular dry-stone masonry using wooden wedges, see Figure D.6. Robert Stevenson & Sons noted the rude workmanship and appearance of great age. In 1811 the outer end of the breakwater was levelled, and the SW pier was badly shattered. Various reconstructions and improvements followed, including chases for stop-logs, blamed by Stevensons' report after 1811 for damage to the SW pier.

**Figure D.6 North Berwick breakwater**
*From Graham (1968)*

**North Queensferry** had at least three piers: Town Pier (lengthened after 1812), and West and East Battery Piers (probably also around 1812).

The harbour at **Pettycur** (Figure D.7) may have begun in 1625, then 'improved' in 1743. The breakwater is formed by a strongly battered face of igneous rock not well suited to fine dressing. Some blocks have been re-used from earlier construction.

**Figure D.7 Pettycur breakwater, damaged crown wall**
*From Graham (1968)*

**Pittenweem** was given the power to build a harbour in 1541, and a breakwater probably existed by 1633, later noted by Adair (1703) and Roy (1747-55). Various piers and basins were built and repaired, and modified from 1721 to 1771. The main East Breakwater is about 220m long, including the pierhead built after 1854, topped by blocks set edgewise. The form of the masonry in the walls varies with square or vertical blocks, or random rubble. A crane at Pittenweem was dated from before 1840.

**Figure D.8 Port Seton, July 2013**

**Port Seton, East Lothian,** was built after 1635 and was in active (commercial?) use until 1810. The original pier appears to have been destroyed. The current harbour (Figure D.8) was formed in 1879-80, and is in active leisure use at present.

**St Abbs, Berwickshire** was established by construction in 1831, principally by breakwaters to NW and another to SE and NE sides. They are formed of large well-dressed, scabbled blocks,[4] dry-set, and accurately coursed.

At **St Andrews**, the main breakwater is set across the original course of the river, supplemented by the Cross-pier. A (fishing) harbour dates from 1222. Early piers were probably of timber with stone filling. They were certainly frequently damaged, including in 1678, 1788, 1816, and 1823 with many requests for funding of repairs. As a consequence, the

[4] Scabbled – dressed to square / rectangular form.

masonry of the breakwater is highly variable, including: roughly-squared and coursed blocks; vertical work; poorly coursed rubble; well-coursed long blocks; vertical blocks founded on a course of boulders; well-dressed and neatly coursed slabs. The construction of the Cross-pier shown in Figure D.9 is substantially more homogeneous than the main breakwater in Figure D.10.

**Figure D.9 St Andrews harbour, September 2009**

**Figure D.10 St Andrews main breakwater showing variable wall construction, July 2003**

**St David's** harbour was built in 1750-60 to ship out coal from Fordell Colliery. By 1836 "*the remains of the old pier showed very poor construction*". By 1847 improvements had "*been made recently*" and the piers showed "*good squared rubble with some rusticated ashlar", "remains of a seawall and parapet … founded on marginal slabs socketed to take rectangular keys … a pillar of stugged rubble … ending in a shallow domed top*".

The harbour structures at **St Monance** enclosing the West and East basins all date from after the first Ordnance Survey of 1854 except the dividing pier, which appears to be enclosed in concrete.

The harbour of **Skateraw, East Lothian**, was constructed by two farmers, Brodie of Thorntonloch and Lee of Skateraw, for the export of limestone and import of coal. It was sheltered by a 85m long breakwater / pier with a minor cross-pier of ~30m. A quay on the SW side carried a crane immediately north of a lime kiln. The breakwater(s) were probably destroyed between 1853 and 1892, now filled by sand / shingle, and "*the breakwater is* *marked by a rickle of debris*". "*The masonry is of long thinnish and well dressed slabs, neatly shaped to curves, but not secured with cramps or keys*". Graham notes two further features on the seaward side of the breakwater below HW. A groyne (or jetty) formed by a double row of large roughly shaped blocks set on edge projects seaward for some 15m. Separately a larger and curved structure, formed by very large blocks set on edge set transversely to the line of the structure, appears to have enclosed a rock-cut pocket. This contrasts with, and does not appear to be connected to, the main breakwater. This breakwater has been discussed earlier in Chapter 2, see also Figure D.11.

**Figure D.11 Remains of Skateraw harbour, from the south, November 2014**

**South Queensferry** has had many piers, certainly since 1693, although the harbour was "*in great measure demolished by a storm*" in 1763. Repair / new construction was then wrecked in 1789, leading to the demolition and re-building of the West Pier in 1795. The harbour was again unsafe by 1815 and a new configuration was designed by Hugh Baird in 1817. This (probably) resulted in the East Pier (104m), Passage jetty, and West Pier (106m). The West Pier shows three different forms of masonry having large blocks to the landward length, then shorter squarer blocks, and then longer thinner slabs. The later construction is used on the cross-pier at the North end. The East pier is faced by large dry-stone blocks, squared but irregularly set, mortared at its end.

Constructed about 1812, the landing jetty is about 105m long and 6m wide. It is paved by setts, larger towards the seaward end keyed together by diamond cut-outs filled by concrete. [NB – Graham uses the imprecise term 'cement'!]

The **Hawes Pier** (285m long and 26m wide) was used by the ferry until 1964, formed of "*masonry is large rubble, well squared and coursed, and with droving*". "*Much of the top is cemented ... marginal slabs with some lozenge-shaped keys of cement*" (sic). Long Craig Pier (350m long) was built in 1812, much resembling Hawes Pier in construction, but does not feature droving, nor the lozenge-shaped keys.

The small harbour at **Starley Burn, Fife**, was (probably) constructed for the export of limestone from the Newbigging quarries, perhaps started before 1792. Surveyed by Ordnance survey in 1854 it had two piers, although these have since been significantly modified by construction of the railway embankment. The outer face of the East pier is formed by very large rough blocks (local igneous rock), and the inside face by smaller (still relatively large) blocks, poorly dressed and coursed. The top of the pier has been washed away. Some (re-used?) limestone blocks show traces of scabbling. The West pier is better preserved.

Torry Bay includes **Torryburn and Torry harbours**. The remains of Torry Pier showed in the 1895 map as "*a belt of scattered boulders*" running 137m out from the shore. The footings are of dark sandstone, set partly vertically and partly horizontally, about 4-5m in width. A small breakwater at the Boat harbour (61m long and 27m wide) was faced by well-dressed blocks on both sides. Some of the core contains blocks set upright, some transverse to the direction of the structure.

The Auld Pier appears to have collapsed well before the OS survey of 1895. The remains suggest a pier ending in a square (10m side) pierhead. Foundation stones show some well-dressed blocks fitted neatly together.

A harbour formed by two piers is recorded at **West Wemyss** is recorded by Adair in 1703, Sibbald in 1710 and Roy (1747-55) and lasted until at least after 1806, but now much obscured. The West pier no longer exist, but about 130m of the East pier exists, although much repaired / reinforced.

A most curious omission is the Duke of Buccleuch's harbour at **Granton**, filled here by a supplementary note at the end of this chapter.

## 5 Harbours in mid-East Scotland – Graham (1976)[5]

**Dundee** is primarily an estuary port where siltation and current flows have been of most concern, and consideration of wave action has probably been of fairly minimal importance. A harbour of Smeaton's era (~1770) was described as "*a crooked wall, often enclosing but a few ... craft*", and was liable to silting. Smeaton's recommendations were carried out only in part, but in 1788 a pier extension was added, and a detached breakwater in 1803. Telford supervised re-modelling the inner basins up to 1820. The outer face of the west protection wall used "*hammer-dressed blocks set obliquely without mortar*", with a "*hearting of quarry rubbish*"!

It is probable that the earliest breakwater at **Arbroath** was formed by timber cribs or cages filled by stones, perhaps still existing in 1693. A (probably new) harbour was formed in 1725 with stone piers equipped with an entrance of 9.5m that could be closed by booms operated by a crane. By 1839 the harbour of 1725 was termed the old harbour, but had been enlarged to a design by J Leslie with a new outer harbour and a seawall of red-sandstone ashlar.

By 1847 the small harbour at **Gourdon** had been expanded by a (new?) East pier and a detached breakwater.

The harbour at **Stonehaven** was primarily a river port until sometime after 1794. After 1825, plans by Robert Stevenson led to removal of some rocks and construction of a 152m long pier, but this still lacked a breakwater to cover the entrance. Extensions of piers on either side reduced the entrance gap "*wide enough to admit a ship*".

The port of **Aberdeen** was established in the estuary of the River Dee, "an area of sandbanks and waterlogged marshes", and continued so until Smeaton's great North Pier constructed in

[5] Unlike Graham (1969), this paper describes harbours divided by geographical sections, starting with Dundee to Arbroath.

1773 to some 460m length, a width of ~ 10m, and a parapet level of around +5m above HW. By 1829 this had been extended by ~150m by Telford, together with a new South Breakwater (245m). Graham then loses interest in the breakwaters and turns to discuss harbour appurtenances.[6]

The original port of **Peterhead** was formed by joining the island of Keith Inch to the mainland in 1735, although there are suggestions that there was a harbour from 1587 protected by "a bulwark in the mouth of the haven". In 1815 it was recorded that the "North Quay was built of very rough material, and without cement", but had "*withstood storms undamaged for more than 200 years*", or "*was very roughly built with masses of undressed granite that had never had to be repaired*". This is at slight variance with a suggestion that in 1793, some 300 tons of "*pavement and cribs*" had been "*thrown down into the harbour ... the stones used in the building of the quay had been too small, its West end had not been founded on solid rock, and the hearting and packing had also been very indifferent.*" Works planned by Telford were constructed 1818-1822, and finished by the Stevenson's in 1827.[7]

There has been a harbour of some form at **Fraserburgh** since 1576. By 1830, it was ranked as the best tidal harbour in eastern Scotland, having acquired a North Pier built of granite (blocks) between 1807-11 to plans by Rennie, and a South Pier under Stevenson's direction after 1818.

Work to construct / improve the (original / inner?) harbour of **MacDuff** is recorded in 1791. An outer harbour was attempted about 1822 but was destroyed by a storm, "*although its remains served as a breakwater for the inner works*". The current three basins (probably including the outer breakwater) represent expansion made in 1877 and later.

A "*useful and commodious*" harbour was claimed in the river at **Banff** in 1798. A series of piers and quays were added between 1806 and 1836 with the North pier being "*useful in preventing swell and agitation*".

There has been a harbour at **Portsoy** to export marble since before 1701, having some form of breakwater (bullwork) before 1724. A new harbour was built in 1825-28, but its outer pier was demolished by storms in 1839, leaving only the inner harbour in use.

In 1847, **Cullen** was classed as a "*small pier harbour*", but was rather changed by improvements of 1886-87 with varied masonry. An iron capstan of 10 bars is dated at 1848.

The origins of **Buckie harbour** are obscure although local fisheries were recorded from 1723. No definitive date is given for port construction, although some had been built before 1842 when Buckie had 117 large fishing boats.

**Portgordon** – breakwater damaged in December 2013, to be repaired October 2014.

Being originally a river port, the history of **Lossiemouth** was primarily marked by movement of the sandbar at the mouth of the river Lossie. The site of an Old Pier on the right bank of the river is now hidden in the sand dunes. A quay with a short pier was constructed on the left bank. In however 1834 a new harbour was started on the present site, known as **Brandenburgh**, with two linked basins quarried out of the rock, with the debris used to form

[6] See Turner (1986) for more on Aberdeen.
[7] See also Buchan (1984) discussed in Chapter 13.

the main breakwater to the north. By 1842 this had established steamer traffic from Leith and London.

The harbour at **Hopeman** may have been formed on a rocky outcrop before 1835 to ship out rock from a nearby quarry. The main pier was breached in a storm in winter 1844-45, perhaps leading to improvements in 1865, 1868 and 1901.

At **Burghead**, a "*good harbour*" was built in 1809 with "*well-squared unmortarted blocks with a moulded string-course some 4ft below the top of the wall*". At LW, the blocks were set vertically, as also at Portsoy. The breakwater was extended SSW in 1834, and again in 1839.

The river harbour at **Nairn** has required jetties to exclude sands from blocking the entrance. A breakwater is recorded as being built in in 1835, and still under construction in 1847. The contractors (Leslie & Mitchell of Inverness) recommended timber crib-work filled by stone to avoid high costs of founding a masonry structure.

Graham adds a commentary on types of construction within which he notes the use by 1400 of timber cribwork, but of hewn stonework by 1526, and of piers using coursed stone by 1579. He outlines the frequency of siltation and sandbars as the cause for harbour failure.

## 6 Harbours in Scotland Highlands and Islands – Paxton & Shipway (2007a)

This listing for ICE Historical Engineering overlaps with those by Graham.

**Rothesay Pier, Bute** (HEW 2447) - Masonry pier, 1752-1781 (and later)

**Hynish Dock, Tiree** (adjunct to Skerryvore lighthouse, HEW 2456) by Alan Stevenson

**Kelly's Pier, Bonawe,** Loch Etive (NN 0070 3219), Drystone, 1752-53

**Arbroath Harbour** (NO 6418 4044), 1841-1846, James Leslie, rubble-filled, ashlar faced seawall. Earlier harbour by John Gibb.

**Johnshaven Harbour** (HEW 2506), 1722 onwards

**Gourdon Harbour** (HEW 2507), 1818-19, masonry rubble-filled pier by James Farquar (of Hallgreen).

**Stonehaven Harbour** (HEW 2509), 1825-26 enclosing pier, Robert Stevenson, Stevenson T (1864).

**Aberdeen Harbour** (HEW 1377), Smeaton, Telford, James Abernethy.

**Aberdeen North Pier** (HEW 1377/01-03)

Phase 1, Smeaton 1774-80, resident engineer John Gwyn, cost £16,000. Walls of horizontally course small squared blocks, internal stone hearting, 1200ft long.

Phase 2, Telford extension, 1811onwards, resident engineer John Gibb, further 900ft, granite blocks laid diagonally over rubble infill, cost £66,000.

Phase 3, further 500ft, resident engineer W Dyce Cay, mass concrete

**Aberdeen South Pier** (HEW 1377/04-05)

First version, 800ft, 1812, supervised by Gibb in consultation with Telford.

Second version, 1050ft, 1870-73, WD Cay, mass concrete onto concrete-filled bags (up to 100 tons). Note use of suit divers preparing seabed and assisting block placement, and use of 25t steam crane (from Cay, 1874, proc ICE Vol 39).

**Peterhead harbour** (HEW 0127), agglomeration of three small harbours (North, South, and Port Henry) and the Bay of Refuge). Input from Smeaton, Rennie, Telford, the Stevensons, Sir John Coode, Thomas Meik.

South Breakwater, 1892-1912, Sir John Coode, 2700ft, 40t concrete blocks onto low height tipped stone foundation mound . Note damage in 1928 moved 34ft by 2 inch.

North Breakwater, 1912-1956, 40t concrete blocks in courses onto levelled rock.

**Fraserburgh Harbour** (HEW 2518)

North Pier (1807-1811) and South pier (1818-1821) to designs by Rennie and Stevenson.

Balaclava Pier (1850-57), design by John Gibb

Balaclava Breakwater (1875-1882), design by James Abernethy, 860ft, cost £60,000, resident engineer JH Bostock, used large bags filled by concrete from a hopper barge to form the lower part up to low water (ascribed to WD Cay) topped by mass concrete (1 part cement to 4.5 part sand/gravel).

**Macduff Harbour** (HEW 2519)

Breakwater, circa 1830, modified in 1877-1878 by D & T Stevenson.

**Banff Harbour** (HEW 2185)

Original harbour enlarges in ca. 1770-1775, Smeaton. Revised / extended by John, James & William Smith, 1818-1828, resident engineer John Gibb.

Damage in 1837 repaired by James Bremner using masonry blocks of 15-40t.

**Portsoy Harbour** (HEW 2521)

East Harbour refurbished, 1882-1883, by James Brand of Glasgow, under direction of John Willet, constructed in mass concrete (1pt cement to 6pt sand/gravel), used 3t steam derrick, cost £9000.

**Cullen Harbour** (HEW 2186)

North Pier built 1817-1819 by William Minto to design by Telford, coursed masonry with rubble infill, but demolished in 1835. North Pier extended in 1886-1887 by RC Brebner supervised by John Willet, using mass concrete walls filled by rubble.

**Buckie Harbour** (HEW 2523)

First harbour around 1855 to design by D & T Stevenson. Extension / revision in 1874-1880, mass concrete, funded by John Gordon of Cluny, designed by WD Cay, detailed by David Cunningham, resident engineer James Barron. Outer concrete 1:4 mix, inner fill 1:12.

**Branderburgh Harbour, Lossiemouth** (HEW 2528)

Constructed by James Bremner 1837-1839, supervised by Alexander Gibb. Excavated basins protected by 'seawalls' using large blocks of masonry.

**Burghead Harbour** (HEW 2530)

First harbour by Andrew Forsythe, mason of Elgin, 1807-1812, extended breakwater by 1835.

**Clachnaharry sea-lock** (HEW 0084/01)

NB Completed in 1812 using a 9hp Boulton & Watt steam pump to drain the lock basin for construction.

**Corpach sea-lock** (HEW 0084/08)

Completed in 1808-1812 using a steam pump to drain the lock basin for construction. Contractors were Simpson & Wilson, supervised by John Telford to 1807 and then by Alexander Easton.

**Fortrose Harbour** (HEW 2108)

Designed by Telford, constructed by John Watson, completed in 1817, cost £4015. Masonry blocks with rubble in-fill.

**Lybster Harbour** (HEW 0404)

Improved ca. 1852 by Christopher Moses of Perth, supervised by Joseph Mitchell followed by D & T Stevenson from 1851. This is where Thomas Stevenson first measured wave heights systematically.

**Wick Harbour** (HEW 0599)

New harbour constructed by James Bremner, completed in 1831, expanded by D & T Stevenson by construction of a new south breakwater from 1863. Stone blocks, 5-10t set on edge onto a rubble foundation 18ft below low water. Reached 100ft by 1870 when the outer third was destroyed. Repaired but a concrete monolith of 1350t was carried away in a further storm in 1877. [Discussed at length by Paxton (2009) and Allsop & Bruce (2020)].

**Reay Harbour** (HEW 1672)

Small harbour completed ca. 1835 for £3000, two piers by James Bremner using (traditional Caithness) near vertical 'flagstones', apparently open-jointed.

**Muckle Flugga Lighthouse** (HP 6066 1977)

Designed by D & T Stevenson, resident engineer Alan Brebner, constructed 1855-1857. Used 10hp steam engine to haul materials up a temporary railway.

# 7 Harbours in Scotland Lowlands and Borders – Paxton & Shipway (2007b)

This listing for ICE Historical Engineering overlaps in places with those by Graham.

**Port Logan** (HEW 1277)

A small pier existed here in 1755, but was destroyed before 1791. Rennie reported in 1831 on the potential for a port to Ireland. The pier of 540ft was built by Colonel McDoull, probably to the layout suggested by Rennie's report. Mole and lighthouse were stabilised / repaired in 1980s, see Graham (1979). Later restoration is shown in Figure D.1.

**Figure D.1 Repaired mole and lighthouse at Port Logan (W. Allsop, 2017)**

**Portpatrick Harbour** (HEW 1276)

Smeaton designed two breakwaters, the southern of which was completed in 1778, but the northern was abandoned in 1801. Rennie revised the designs in 1818, work started in 1820. Around 700+ labourers completed the southern breakwater (again) in 1836, but it was damaged in 1839. The north pier was unfinished. Portpatrick Railway Act in 1857 allowed the railway to arrive by 1862. Harbour was abandoned in 1873. Lighthouse was dismantled and re-used in Columbo, Sri Lanka in 1871.

**Eyemouth Harbour** (HEW 2399)

A river mouth harbour, long requiring a training mole to channel the river, and reduce siltation, usually a sand bar across the river. Old Pier was constructed in 1747 by William Crow, but the harbour was damaged in 1767 (and again in 1794). Smeaton designed North Pier, 1769-1773, using inclined courses, some still visible.

**Dunbar Harbour (or Lamerhaven)** (NT 6814 7922)

Extremely complicated headland harbour with two basins connected / divided by a causeway and small (low and narrow) bridge. East Pier of horizontally coursed rectangular blocks probably built in 1717.

**Cockenzie Harbour** (HEW 1380)

Harbour originally started in 1630 for export of coal and salt, rebuilt 1833-1834 to plans by Robert Stevenson & Sons. Main (east) breakwater 227ft of squared sandstone blocks and west breakwater of similar construction.

**Leith Docks (Edinburgh)** (HEW 0080)

Series of locked basins developed over many years. No major breakwater, but a 4400ft long seawall by Peter Whyte formed by large fitted concrete blocks on a 1:1 slope, backed by "hand-packed rubble" and a puddled clay core. [8]

**Troon Harbour (Ayrshire)** (NS 3098 3148)

Developed on / behind a natural rock promontory, with piers developed by the Duke of Portland from 1808 including the lee (eastern) breakwater, and some reinforcement of the rock promontory in the Western breakwater / seawall.

**Ardrossan Harbour (Ayrshire)** (HEW 1035)

Like Troon, also on a rock promontory. Curved southern pier 2700ft constructed by 1809 (Cost £70,000 against original budget of £40,000). In 1815 Rennie estimated that a further £90,000 was needed. An outer basin protected by 1320ft of new breakwater were added in 1886-1892 engineered by Robert Robertson.

**Charlestown Harbour (Fife)** (HEW 1623)

Harbour for the transport of lime and/or coal, pier formed by un-mortared squared sandstone blocks. Outer basin piers added later, north-west pier around 1840 and the south-east after 1853.

**Burntisland Harbour (Fife)** (NT 2302 8541)

Formed behind a rock promontory, the West Dock opened in 1876. Served for the train ferry from Granton 1850-1890 when superseded by the Forth Bridge.

**Anstruther Harbour (Fife)** (NO 5683 0341)

The original harbour from before 1703 was extended in 1753 by constructing the West pier of dry-stone blocks. This was extended between 1866 and 1877 by a further 470ft of western breakwater and east pier, 960ft, engineered by D & T Stevenson. RLS noted the *'green glimmer of the divers' helmets far below'*

**Crail Harbour** (NO 6216 0741

West pier built 1826-1828 by John Gosman to design by Robert Stevenson. Some sections use vertically placed (dry) masonry.

## 8 Scottish Fishing Harbours – Willet (1886)

Development of herring fisheries along the Forth to Cromarty coasts over the previous 30 years had stimulated / accelerated development of Harbour Board and private harbours.

Fishing boats based at Fraserburgh harbour expanded from 389 in 1854 to 742 in 1885. The harbour faces mainly East. Three basins (North, South and Balaclava) were protected by four piers, and then by a new outer breakwater. The third harbour (Balaclava) was constructed in 1851, being rather better protected against waves than the older two harbours. Abernethy

[8] Again, nothing on Granton Harbour. Western or Eastern Breakwaters, built for the Duke of Buccleuch and opened by Queen Victoria in June 1838. Both breakwaters are each over 3,000 ft long, but see supplementary note at end of this chapter.

(past President ICE, 1881-1882) advised the Harbour Commissioners in 1857 to extend the Balaclava Pier out east to form a breakwater. In 1872, Willet urged the adoption of this design which was started in 1875 by widening the existing Balaclava pier to carry construction traffic whilst minimising disruption to the working quays. The lower part of the new breakwater used concrete in 28-50t jute canvas bags, placed (as at Aberdeen) using a bottom-opening hopper barge. Interestingly, the barge was *"...warped to the place of deposit ..."* so no use of steam tugs. Concrete mixes varied between summer (milder waves) and the " *... more stormy season*" being 1 part cement to 9 parts of sand / gravel / aggregate versus 1 part cement to 7 in stormy conditions. Generally beach sand and shingle were used, sometimes using crushed stone and sand. The breakwater (270m long) was started in spring 1878 and finished in autumn 1882. It used 11,700m3 of concrete in bags, and 19,200m3 of concrete in formwork (bays of 4.9m) at an annual rate equivalent to 6120m3/yr, at best placed at 306m3 in an 8 hour shift. Two casting shifts were possible each week. Portland cement (9750t) cost £1.85-2.2 per ton.

The works were damaged in five places during the construction period, mostly between LW and HW, and the parapet wall was carried away in another place. In March 1883 a breach occurred at a junction in the superstructure attacking loosely compacted concrete and blasting through the roadway surface. The same storm broke off a 10m length of parapet (1m x 1m). At some points moving rocks against the seaward face caused abrasion damage, at others weaknesses at junctions allowed the concrete to be eroded. The cost of repairs reduced in time reaching a total of 1.75%.

Sandhaven harbour is 5-10km west of Fraserburgh, facing North-East. An existing breakwater arm completed between 1837 and 1841 enclosed about 0.04km2. A new easterly pier was intended to enclose an additional 0.2km2, formed by mass concrete walls in forms.

## 9 North-East England – Rennison (1996)

Another compilation of Engineering Heritage for ICE.

**Seahouses Harbour** (HEW 1988)

Inner harbour of masonry around 1786 by Robert Cramond of Sunderland. Extended in 1899 by Sir John Coode and J Watt Sandeman by 950ft long rubble-faced (vertical?) north pier, east breakwater in mass concrete

**Warkworth Harbour, Amble** (HEW 1990)

Coal exporting port formed on River Coquet by formation of 2300ft long sloping face masonry north breakwater, completed in 1849 at cost of £100,000. Branch railway completed by 1854. North breakwater extended in early 1900s and coal shipments rose to 500,000+ tons / annum in 1914 (and then declined?).

**Blyth Harbour** (HEW 1992)

North Dyke, a rough mound of stones, was formed in 1756. In 1814, John Rennie recommended an outer breakwater, but this was only started by James Abernethy in 1854. Blyth Harbour and Dock Co. was replaced by Blyth Harbour Commissioners in 1882, appointing J Watt Sandeman as engineer. Around 1900 the North Breakwaters was extended to 4600ft with a new lighthouse built by 1907.

**Seaton Sluice** (formerly Hartley Harbour) HEW 987

Small natural harbour protected in 1661 by a short pier constructed by Sir Ralph Delaval, Sluices to flush out silt at LW constructed ~1675. Sir John Hussey Delaval then cut a new gated easterly entrance through the headland, now blocked by rock rubble. The port declined after closure of Delaval Colliery in 1862.

**River Tyne Piers** (HEW 759)

James Walker designed two piers to protect the river mouth, constructed from 1855 by Benjamin Lawton, supervised by Philip John Messent as RE. From 1862, Lawton's contract was terminated, works continuing by the Tyne Improvement Commission still supervised by Messent, using 36-40t concrete blocks made by Messent's concrete mixing machine. Works completed by 1895, but substantial damage caused to the North Pier in 1895-97. This was then rebuilt on a new line by Sir John Jackson Ltd under supervision of Sir John Wolfe Barry and Coode, Son & Matthews, completed in 1908.

**River Wear Piers (Sunderland Outer Harbour),** (HEW 1860)

New north and south breakwaters of masonry faced concrete, founded on 50-120t jute bags filled by concrete, built by direct labour under control of Henry Hay Wake, Engineer to the River Wear Commission. Foundation taken to 18in above LW. North Pier 2880 ft long, formed of 43t blocks, built 1885-1903. South Pier is 2666 ft long, built 1891-1914.

**Seaham Harbour** (HEW 758)

The main inner harbour was constructed by direct labour supervised by William Chapman for the Marquess of Londonderry, completed in 1831. The outer breakwaters were constructed in concrete masonry (1383 and 878 ft long) by S Pearson & Sons Ltd, 1899-1905, to a design by Henry Hay Wake and Patrick Walter Meik.

**River Tees Breakwaters** (HEW 2005)

The North Gare and South Gare breakwaters were built from 1863 using iron ore slag in 2-3t blocks faced by a mass concrete wall on the seaward side. The outer ends of the breakwaters were protected by 30-40t random concrete blocks

**Silloth** Docks (HEW 2012); **Maryport** Docks (HEW 965); **Workington** Harbour (HEW 2014). - No significant information on their breakwaters.

**Whitehaven Harbour** (HEW 966)

One of the oldest harbours on this coastline, Henry II granted the port in the 12th century to the Prior of St Bees. A new West Pier was proposed by Joseph Whidbey and Sir John Rennie in 1823, built by Fox 1824-1830. Rennie also proposed / designed a new North pier, constructed by David Logan, 1834-1841, x-ref discussion by Rennie to paper by Scott Russell (1847).

**Whitby Harbour**, (HEW 2020)

Each of the two breakwaters is formed by inner and outer parts. The two outer parts, each 500ft long, were constructed by J. Watt Sandeman & Son from 1905. They are each separated slightly from their inner parts by a small gap (~10-15m). The inner parts of the West Pier were started after 1735, and appear to have been progressively extended / reconstructed under supervision of Jonathan Pickernell to 1812, then James Peacock, and then Francis Pickernell to 1861. [Yorkshire Ports website notes: the West lighthouse was built in 1835 and the East in 1855.]

**Scarborough Harbour** (HEW 933)

Protected by two piers in 1734, later acquiring a third pier (West, East and Vincent's).

[Yorkshire Ports websites note that the West Pier was completed shortly after 1325, the Vincent Pier (sometimes also termed Old Pier) in 1752, and the East Pier in 1811 (although this was re-armoured with rubble and Accropodes ~ 2005)].

**Queen's Pier, Ramsey, Isle of Man** (HEW 1001)

Truly a pier, not a breakwater, 2150ft long, built in 1882-86, by Head Wrightson at a cost of £45,000.

## 10 South-West England – Otter (1997)

**Smeaton's Pier, St Ives** (HEW 1397)

Coursed stone masonry walls battered at 2:1 retaining rubble infill, founded on a rock mound on dense sand. Constructed 1767-1770 by Thomas Richardson (previously Smeaton's mason on the Eddystone lighthouse.

**Charlestown Harbour, St Austell** (HEW 139)

Small harbour protected by a masonry breakwater, with a locked inner basin, constructed 1791-1800, designed by Smeaton.

**Bude Canal Sea Lock** (HEW 1066)

Sea lock enlarged and protected by stone and rubble breakwater wall in 1835 by James Meadow Rendel for 300t ships.

**Plymouth Breakwater** (HEW 126)

Designed / supervised by John Rennie (senior) with Joseph Whidbey, and then by Sir John Rennie, this 5100 ft long rubble mound is paved by limestone blocks over its upper part. Started in 1812 and providing useful shelter by 1814, it still required stone dumping (3.6 million tonnes by 1847). See 2.10.

**Brixham Breakwater** (HEW 1487)

Designed by James Rendel, 1400ft built in 1843. Extended by further 600ft in 1909, and further 1000ft 1912-1916. Limestone faced by jointed masonry.

**Haldon Pier, Torquay Harbour** (HEW 1640)

Concrete blocks (160ft3), placed by divers on broken stone levelling in 1870.

**Portland Harbour** (HEW 124)

The harbour is formed by four breakwaters, the two southern being designed in 1849 by James Rendel as a rubble mound *"tipped promiscuously"*. Lower slopes at 1:1.5, upper slopes at 1:6. Construction continued to 1868 supervised by John Coode. The two northern breakwaters were started in 1893. The south entrance was blocked in 1914 by sinking HMS Hood just outside it. Portland harbour was included in the 'Harbours of Refuge' review by Allsop (2020).

**Cobb breakwater, Lyme Regis** (HEW 414)

Some parts date from 1300, modified by William Jessop from 1805, then repaired in 1825.

**Brighton Chain Pier**, demolished (HEW 428)

Formed by four 255ft spans, suspended by wrought iron chain, constructed in 1822-23 by Cpt Samuel Brown (designer), destroyed by a storm in 1896.

**Brighton Marina** (HEW 633)

Two breakwaters formed by cylindrical caissons placed against each other, interlocked by keyways, 1971-1979. Designed by Lewis & Duvivier and constructed by Taylor Woodrow.

**Shoreham Harbour** (HEW 1412)

Formed in the mouth of the River Adur by training walls (1817-21) through the shingle beach. In 1826 Telford added a training pier.

**Newhaven Harbour** (HEW 1414)

The mouth of the River Ouse was stabilised around 1730. John Reynolds constructed (new?) training piers in 1733-35, establishing Newhaven Harbour. New western groyne built by James Walker in 1843 maintained 7ft entrance depth, but replaced in 1878 by the West Breakwater of mass concrete on blockwork.

**Royal Harbour, Ramsgate** (HEW 484)

Completed in 1792 by two breakwaters, 625yd and 520yd, faced by Portland stone, some Whitby sandstone, and granite copes, filled by rammed chalk mixed with gravel and lime mortar. Designed by William Etheridge, later supervised by John Smeaton 1778-1792. The harbour is now enclosed by the rubble mounds of Port Ramsgate.

**Dover Harbour** (HEW 125)

The historical harbour in the River Dour was extended in 1847 by construction of Admiralty Pier formed by 7-8t concrete blocks, followed the Prince of Wales Pier 1892-1902, designed by Coode, Son & Matthews. The Admiralty Harbour, also designed by Coode, Son & Matthews, and constructed by S. Pearson, involved further extending Admiralty Pier (2000 ft), a detached South Island Breakwater (4212 ft), and the Eastern Breakwater (2942 ft). these were formed by 26 and 41t concrete blocks placed on the chalk bed, the intertidal blocks to 4 ft below LW being faced by granite. This harbour is discussed in the 'Harbours of Refuge' paper by Allsop (2020). The stability of the outer (South) breakwater is discussed by Allsop & Bruce (2020b).

**East Pier, Folkestone Harbour** (1446)

Harbour formed by two piers by Telford in 1808 using undressed stone slabs placed at 45° slope. The further East Pier (700 ft) still survives.

## 11 Supplementary note on Granton Harbour

Based on [www.grantontrawlers.com](http://www.grantontrawlers.com) and [www.grantonhistory.org](http://www.grantonhistory.org) and on visits to the site.

With introduction of steam-power more vessels began plying trade into and across the Forth, requiring more landing places for both cargo and passengers. A new harbour was proposed Figure D.1 at Granton, supported by the Duke of Buccleuch who owned the land and foreshore rights. Reports and estimates were approved by a meeting of "Mercantile and Nautical Gentlemen" in Edinburgh, so convincing the Duke of its practicability that he bore the cost of the works. The Duke was advised about the design of the harbour by Robert Stevenson, better known as a lighthouse engineer.

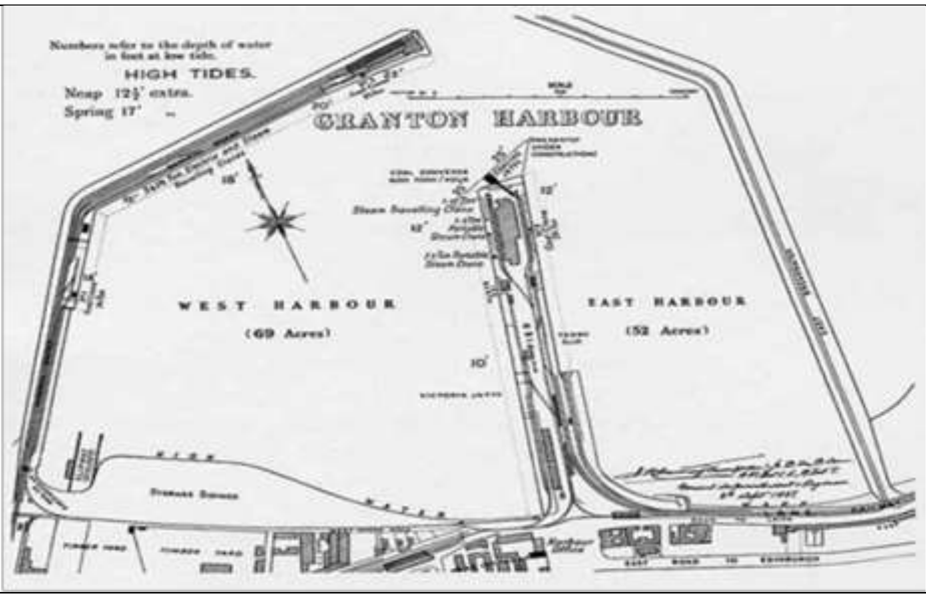


**Figure D.1 Granton harbour built for the Duke of Buccleuch**
*From [www.grantontrawlers.com](http://www.grantontrawlers.com)*

**Figure D.2 Granton East Breakwater**

The bill for the Duke of Buccleuch to make and maintain a pier at Granton was given royal approval in April 1837 by William IV. The pier was constructed by Messrs. John Orrell & Co of Liverpool, the first part of which was opened in June 1838, the date of Queen Victoria's coronation, so was named the Victoria jetty. The last section was completed in October 1844.

In 1842 an act was passed for construction of two breakwaters to provide a safe landing place and harbour, protecting from both easterly and westerly winds.The Western breakwater was undertaken in two sections and the first completed in 1849 at a length of 1500ft. When complete the total length was 3100ft.

**Figure D.3 Granton East Breakwater details, a) edge rail; b) end-on blockwork, 2017**

Work on the Eastern Breakwater (Figure D.2) commenced in 1853 and completed in 1863 at a length of 3170ft. Stone for the breakwaters was quarried at Granton Quarry just west of the Western breakwater. This breakwater is battered on each side, Figure 12.15, with stones closely-fitted, but apparently not cemented, Figure D.3. Along the edge of the crest, a slender iron rail is used to link blocks together giving a tensile restraint against block movement.

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Construction of Granton harbour alerted the railway companies who soon laid tracks along the Western breakwater and down the Middle Pier to carry coal from their Lothian coalfields.

# Appendix E – Biographies

APPENDIX E BIOGRAPHIES

## Preamble

These short biographical notes are intended simply as a handy guide to personae dramatis in the ICE papers and discussion in this thesis. They have been shortened and/or adapted mainly from ICE Obituaries of the time, or from Grace's Guide to British Industrial History (https://www.gracesguide.co.uk ) or Wikipedia. Other sources are listed in the References in the main body of the thesis.

## AIRY, Sir George Biddell (1801–1892)

George Airy was born at Alnwick, Northumberland, in July 1801. His father rose from farm labourer to collector of excise in Northumberland, then Hereford in 1802, and Essex in 1810, but in 1813 he lost this appointment and lapsed into poverty. From 1812 Airy spent holidays at Playford, near Ipswich, with his uncle Arthur Biddell, in whose library he studied optics, chemistry, and mechanics. From 1814 to 1819 Airy attended grammar school in Colchester, then to Trinity College, Cambridge in October 1819 where he graduated in 1823 as senior wrangler and first Smith's prizeman.

In December 1826 Airy was elected Lucasian professor of mathematics at Cambridge. In February 1828 he became professor of astronomy and director of the Cambridge University observatory then astronomer royal for 46 years. He re-equipped the Royal Greenwich Observatory with instruments designed by himself. Airy saw the main work of the Greenwich observatory as construction of accurate stellar / planetary tables and maps to be used by the Royal Navy. In 1859 Airy installed a 13 inch aperture equatorial refracting telescope. In 1838 he created Greenwich's magnetic and meteorological department. One abiding precept was that any astronomical observation that had not been mathematically reduced was useless, so when he started at Greenwich he resolved to publish the backlog of outstanding lunar and planetary observations made between 1750 and 1830. For this he received the gold medal of the Royal Astronomical Society in 1846.

In 1838, he had commenced experiments to study the influence of the hulls of iron ships upon their compasses, and devised a system of correction. He was keen to determine the longitudes of as many places as possible with reference to the Greenwich meridian. Airy's astronomical and geodetic work played a major part in Greenwich being accepted in 1884 as the international zero longitude meridian.

Airy became a universal adviser to the government on matters involving physical science, including weights and measures commission (1838–42), which he chaired, and the tidal harbours commission. He was actively involved in the improvement of lighthouses. He reduced tidal observations of Ireland and India, and was consulted on the launch of the steamship Great Eastern, Charles Babbage's calculating engine, the smoky chimneys of the palace of Westminster, the repair of Big Ben (after the bell cracked in 1859), and the laying of the Atlantic telegraph cable. His published works, including books, official reports, major research papers, authoritative encyclopaedia chapters, exceeded 500 items. Airy turned down three previous offers of a knighthood, in 1835 (on the grounds that he was too poor), in 1847, and in 1863 (because a £30 fee was involved).

## BARLOW Peter (1776-1862)

Born in Norwich in October 1776, Peter Barlow was educated at a local foundation school before becoming a tradesman. After gaining considerable scientific knowledge through self-tuition, he became a schoolmaster. In 1801 he took the post of assistant mathematical master in the Royal Military Academy, Woolwich. Following that, he became professor at Woolwich.

His book, An Elementary Investigation of the Theory of Numbers, was published in 1811, and in 1814 A New Mathematical and Philosophical Dictionary, as was his New Mathematical Tables. In 1817 his Essay on the Strength of Timber and other Materials compiled results of numerous experiments in Woolwich Dockyard, providing much-needed data for engineering calculations. Experiments on the strength of iron forming the basis of the design for the Menai suspension bridge were submitted for his examination by Telford.

In 1819 Barlow started to investigate the phenomena of induced magnetism in order to remedy deviations of the compass caused by increasing quantities of iron in the construction and fittings of ships. He became a member of the Institution of Civil Engineers in 1820, and a fellow of the Royal Society in 1823, then elected onto the council of the Royal Society.

In 1831 Barlow published the results of an experiment displaying the similarity between the magnetic action of the earth and of a wooden globe coiled round with a current-carrying copper wire. Barlow was also very involved with experiments on steam locomotion. He sat on railway commissions in 1836, 1839, 1842, and 1845, and two reports addressed by him in 1835 to the directors of the London and Birmingham Company on the best forms of railway equipment were regarded as of the highest authority.  Barlow resigned his post at Woolwich Academy in 1847.

## BIDDER, George Parker (1806-1878)

George Bidder was an English engineer and calculating prodigy. Born in Moretonhampstead, he displayed a natural skill at calculation from an early age. His father, a stonemason, exhibited him as a "calculating boy", but neglected his general education until Sir John Herschel. arranged that George was sent to school in Camberwell. He did not remain long as his father wished to exhibit him again, but he was enabled to attend classes at University of Edinburgh through Sir Henry Jardine after whom Bidder later founded a "Jardine Bursary.

On leaving college in 1824 he received a post in the Ordnance Survey, but gradually moved into engineering work. In 1834 Robert Stevenson, who he had met in Edinburgh, appointed him to the London and Birmingham Railway. In 1835, Bidder assisted George Stevenson in his parliamentary work, including schemes for railways between London and Brighton and between Manchester and Rugby via the Potteries. He was introduced to engineering and parliamentary practice at a period which saw the establishment of the main features and principles of English railway construction.

Bidder has been praised as the best witness that ever entered a committee-room. There were few important engineering projects brought before parliament in which his services were not secured by one party or the other. In 1837 he was engaged with R. Stevenson in building the Blackwall railway, and he designed a method to disconnect a carriage at a station while the rest of the train went on without stopping.

For half a century he attended the weekly meetings of the Institution of Civil Engineers, of which he was elected president in 1860. His advice was frequently sought by the government on both of naval and military engineering.

## BRUNLEES, Sir James (1816–1892)

James Brunlees was born at Kelso, Scotland. His father was gardener and steward to the duke of Roxburghe's agent. Brunlees was educated at the parish school and afterwards at the local private school. At 12 he worked as a landscape gardener; but he had a natural taste for engineering. James Brunlees met the civil engineer Adie, on the Roxburghe estates, and was employed to make a survey of the estates. He saved money to pay for classes at Edinburgh University, where he studied for several sessions.

In 1838 Adie took him as pupil and assistant on the Bolton and Preston Railway line. He subsequently carried out surveys for Joseph Locke and John Edward Errington on the Caledonian line to Glasgow and Edinburgh, before becoming an assistant to John Hawkshaw on the Lancashire and Yorkshire Railway.

In 1850 Brunlees set up his own practice, becoming engineer to the Londonderry and Coleraine Railway in Ireland, requiring construction of embankments across the River Foyle. The success here led to his appointment as engineer to the Ulverston and Lancaster Railway which involved embankments across Morecambe Bay, and iron viaducts on screw-pile foundations across the Leven and Kent estuaries. Brunlees acknowledged the help of Harry Brogden in developing a method of sinking piles using water jetting, described in ICE papers which helped consolidate his reputation.

Brunlees was based in Manchester, but in 1856 he was appointed engineer to the São Paulo Railway in Brazil, and shortly afterwards he moved to London where he remained for the rest of his career. The São Paulo Railway crossed the steep slopes of the Serra do Mar, and he had to adopt a system of inclined planes and stationary engines, described in an ICE paper by the resident engineer, D. M. Fox. Other work in South America included the Central Uruguay and Hyugentas Railway, the Bolivar mineral line in Venezuela, and a 400 ft long iron pier in the River Plate.

He designed many iron structures for tidal waters through his career, including examples at Llandudno, New Brighton, Southport, and the longest at Southend. His most important dock scheme was at Avonmouth (1868–75). He was engineer for several railway lines in Lancashire and Cheshire including the tunnel under the Mersey between Birkenhead and Liverpool. He was joint engineer with Charles Douglas Fox, and on the completion of the work in May 1886 they were both knighted. He was also, with Hawkshaw, engineer to the original Channel Tunnel Company.

Brunlees became a member of ICE in 1852 and was president 1882–3. He was also a member of IMechE from 1870, and a fellow of the Royal Society of Edinburgh.

## BURGOYNE, John Fox Field-Marshal Sir, (1782-1871)

John Fox Burgoyne was born in Soho, London in July 1782, son of General Burgoyne who commanded the expedition from Canada against the United States in 1777, and who was forced to surrender with his army to the Americans at Saratoga. John Burgoyne was 10 years old at his father's death, and his father had left debts which the proceeds of his estate barely covered. Lord Derby, close friend of General Burgoyne, took care of the orphan, giving him a home and education. In 1796, he entered the Royal Military Academy at Woolwich; and in 1798, at age 16 obtained his first commission as lieutenant in the Royal Engineers. John Burgoyne then commenced a career of active service, which continued without intermission for seventy-one years.

In 1800 he was employed in the blockade of Malta, and capture of Valetta. He joined the army in Sicily in 1806, accompanied the expedition to Egypt, as Commanding Engineer, and served at the assault of Alexandria, and siege of Rosetta. Having impressed Sir John Moore, in 1808 he accompanied the expedition to Sweden, and afterwards to Portugal. He was at the retreat to Corunna, during which he blew up the bridge at Benevente in front of the advancing enemy, delayed their pursuit.

In 1809, he joined the army in Portugal under the Duke of Wellington, and was engaged in all the great actions of the campaign, including the retreat at Torres Vedras, where he again blew up Fort Concepcion in the presence of the enemy.

He served as Commanding Engineer at the siege and capture of the Forts of Salamanca and in the battle of Salamanca, at the capture of Madrid and the Retiro, where two thousand French troops surrendered, and at the siege of Burgos, where he was wounded. He was present at the battle of Vittoria, where his horse was shot under him, and at the siege of San Sebastian, where he was severely wounded. He conducted the siege of the Castle of San Sebastian as Commanding Engineer. Burgoyne afterwards accompanied the expedition to New Orleans as Commanding Engineer, and served in the capture of Fort Bowyer. On return from New Orleans, he joined the army of occupation in Paris.

At the close of the war in 1815, Lieut.-Colonel Burgoyne, aged 32, had been mentioned eight times in dispatches, had received five gold medals, the cross of the Tower and Sword, and the decoration of the Bath. During the campaign in the Peninsula, Burgoyne earned that high reputation for bravery, judgment, and unflinching adherence to duty which accompanied him throughout.

After Waterloo, was appointed Commanding Engineer at Chatham until 1827, when he was sent to Portugal with the army under Sir William Clinton. On his return, he was Commanding Engineer at Portsmouth. In 1831, he was appointed chairman of the Board of Public Works in Ireland, which he held for 13 years. During this he acted as chairman of commission to improve navigation of the Shannon.

He was a member of the Commission in 1836 “to consider a general system of Railways for Ireland,” of which the late General Sir Harry Jones was Secretary. Colonel Burgoyne was also Chairman of Commissioners of Drainage, Member of the Board of the Wide Streets Commissioners for Dublin, and chief Commissioner of Kingstown and Dunmore Harbours. Colonel Burgoyne was one of the founders and the first President of the Institution of Civil Engineers of Ireland, and delivered the inaugural address on 6th August 1835.

In 1838, he was promoted to Major-General, and was made Knight Commander of the Order of the Bath. In February 1839, Sir John Burgoyne was elected an Honorary Member of The Institution of Civil Engineers.

In 1845, Sir John Burgoyne was offered the post of Inspector-General of Fortifications. Soon after his new position, he was called to organize and direct the Relief Fund for Ireland, caused by the famine in 1847. On his return to England he was employed on many Commissions connected with the public service. In 1849, he was ordered to inquire into the state of the Caledonian Canal, and causes of inundations of Inverness.

Sir John Burgoyne was nominated one of the original members to the Commission of Metropolitan Sewers and then re-appointed to the second Commission. In 1854 Sir John was sent to consider the defence of Constantinople and the Dardanelles. Shortly afterwards he was appointed a member of the Defence Committee, on which he continued until his death. He was one of the Jurors of the Great International Exhibitions of 1835 and 1862. After an explosion of a magazine at Erith in 1864, he was appointed President of the Committee to inquire into the state of Military and War Department Magazines. The report drawn up by him became the textbook by which storage of gunpowder is regulated. In 1865, he was made Constable of the Tower of London and Lord Lieutenant of the Tower Hamlets. In 1868, Sir John Burgoyne retired from the office of Inspector-General of Engineers, but considering his long services, extending without break over seventy-one years, he retained the full salary of the office, and he was promoted to Field-Marshal.

## CAY, William Dyce (1838-1925)

William Cay trained as a civil engineer with Professor James Thomson (Belfast) and B & E Blyth in Edinburgh where he worked on Portpatrick Railway, Tay and Forth bridges. In 1862-63 he was Resident Engineer on the Turin and Savona railway, then in London for 2 years, returning to Edinburgh Waterworks (1865-67). His "most important civil engineering work" was as Resident Harbour Engineer at Aberdeen for some 13 years (1867-80), where he directed harbour improvements, including the new South Breakwater, and the outer section of the North Breakwater, extending previous sections by Smeaton and Telford. He devised a method to place concrete in large bags (up to 100t) through a bottom-opening barge, for which innovations he received medals in 1887 and 1888. His method to form breakwater foundations were used at Buckie, Lerwick, Fraserburgh, Arbroath and Newhaven, and at New Plymouth (New Zealand). He was elected MICE in 1872 and practised as a consulting engineer in Edinburgh (1880 – 1907) and London (1909-1919). Cay never married. He died in Folkestone in 1925.

## COODE, Sir John (1816–1892)

John Coode was born at Bodmin, son of a solicitor. He was educated at Bodmin grammar school then entered his father's office. His natural tastes were for engineering, so he was articled to civil engineer James Meadows Rendel of Plymouth. On completion of his pupillage he continued briefly with Rendel and subsequently on the Great Western Railway between Bristol and Exeter.

Between 1844 and 1847 Coode had his own practice in Westminster as a consulting engineer, working predominantly on the Santander to Madrid Railway. In 1847 he was appointed resident engineer in charge of construction of Portland harbour, which had been designed by Rendel, on whose death in 1856, Coode was appointed engineer-in-chief, a post retained until end of the work in 1872. This was the largest deep-water artificial harbour in Great Britain, and was of major national importance at the time, constructed partly by convict labour. John

Coode was knighted in 1872 for his services. In 1856, Coode had established his firm of consulting engineers which survived, with amalgamations, through three generations of his direct descendants. From 1858 he served as a member of the Royal Commission on Harbours of Refuge around Britain and Ireland. He also began to develop his overseas work being consulted by several colonial governments and the Crown Agents on harbour works, and he made several journeys to South Africa, Australia, and India. Following his appointment as engineer-in-chief for Table Bay harbour, work proceeded from 1859 to 1882. In 1873, he reported on the harbour for Colombo; construction of this major harbour started in 1874, and the works, extended with increasing trade, were completed in 1885. In the late 1870s he designed works for Port Natal, Durban; advised on harbour works for Mossel Bay, Knysna, and Plattenberg Bay in Cape Colony. In 1878 he recommended harbour improvements for Port Phillip, Melbourne, where 'Coode island' results from realignment of the River Yarra. He also advised the state of Victoria on several other harbour proposals and river improvements. He inspected major and minor harbours in New Zealand, leading to recommendations for works undertaken at Dunedin.

In 1885 Coode inspected sites for port works at Trincomali, Bombay, and Singapore, selecting the latter for a new graving dock. In the same year he gave comprehensive advice for port developments for New South Wales. He also advised on harbour proposals for St Lucia, Trinidad, Accra, Lagos, Kyrenia, Penang, Sierra Leone, Heligoland (a British colony), Newfoundland, Pondoland, Fremantle and Port Adelaide. Among other harbour works for which Coode was responsible were Waterford harbour, and plans for Dover commercial harbour, work on which was proceeding at the time of his death. He was a member of the royal commission on metropolitan sewage discharge (1882–4), and of the international commission of the Suez Canal until his death in 1892. He was made KCMG in 1886.

Coode was elected a member of ICE in 1849, served for many years on Council, and was President from May 1889 to May 1891. He was also an active member of the Royal Colonial Institute, and sat on its council from 1881 until his death. He wrote many professional reports about the harbour projects. Coode died at Brighton on 2 March 1892.

## HARRISON, Thomas Elliot (1808–1888)

Thomas Elliot was born in April 1808 in Fulham, Middlesex, son of William Harrison, an official at Somerset House. Shortly after his birth his father moved to Sunderland in order to establish a shipbuilding firm and promote local mineral railways. Thomas was educated in Durham, thence apprenticed to William & Edward Chapman, civil engineers of Newcastle, where he gained experience in dock construction for the coal trade.

Harrison's first connection with railways came as an assistant to T. L. Gooch (1808–1882) who was surveying part of the route of the London and Birmingham Railway for Robert Stevenson (1803–1859). In 1832, with the support of Stevenson as consulting engineer, Harrison became engineer of the Stanhope and Tyne Railway and, two years later, of the Durham Junction Railway. Victoria Bridge over the River Wear was erected to Harrison's. This experience qualified him for the role of resident engineer under Robert Stevenson completing the east coast rail route through construction of the Newcastle and Darlington Junction and Newcastle and Berwick railways. According to Stevenson, Harrison carried the main responsibility for building the lines and the high level bridge across the Tyne and the Royal Border Bridge, Berwick.

With the amalgamation in 1849 of these and other lines, Harrison was appointed chief engineer and general manager of North Eastern Railway in 1854, for which he was made chief engineer for the rest of his life. Harrison played a significant part in creating the largest railway monopoly through negotiations and parliamentary enquiries that accompanied the North Eastern's crucial mergers. The North Eastern was among the slowest to replace iron rails with steel, but under Harrison's guidance the North Eastern was in the vanguard in using the Smith vacuum and Westinghouse air brakes. He demonstrated the advantages of hydraulic power for the gates of Tyne Dock, Jarrow, and the swing bridge over the River Ouse between Hull and Doncaster.

Harrison sat on the royal commission on water supply, 1867–9, and gave evidence to the royal commission on railways, 1865–7. He was consulting engineer of the London and South Western Railway and was much in demand as an arbitrator in disputes between railway companies. He valued above all other honours his election as President of ICE in 1873. He died at Whitburn, near Sunderland, in March 1888 whilst still heavily involved in designs for the Forth Bridge.

## HAWKSHAW, Sir John (1811–1891)

John Hawkshaw was born in April 1811 at Leeds, fifth of six children of Henry Hawkshaw (1774–1813), publican of Leeds, and his wife, Sarah Carrington. After attending Leeds grammar school, John became pupil of Charles Fowler, a local road surveyor, working on turnpike schemes. In 1830 he became assistant to Alexander Nimmo (1783–1832) and helped survey a railway from Liverpool to the Humber via Leeds.

In July 1832 Hawkshaw became engineer to the Bolivar Mining Association in Venezuela, where he lived until mid-1834, when ill health forced him to return. Back in England, Hawkshaw worked initially for Jesse Hartley, and subsequently for James Walker. For Walker he surveyed the Leipzig–Dresden Railway, and the Hull and Selby Railway, giving parliamentary evidence in March 1836.

In 1836 he was appointed engineer to the Manchester, Bury, and Bolton Canal and Railway, and moved to Manchester, where he lived until 1850.

In 1838 Hawkshaw wrote his Report to the Directors of the Great Western Railway, which was critical of the broad gauge, which brought him to national prominence. In 1845 he advocated a light rail system with frequent services in preference to trunk routes with infrequent express trains.

In 1845 he became engineer to the Manchester and Leeds Railway with which he was to remain associated until his retirement. The Pennine lines involved major heavy civil engineering works, numerous tunnels, and masonry viaducts. Hawkshaw was also responsible as railway engineer for many iron bridges and viaducts, including the Junction Railway in Salford, with a viaduct on iron columns above the street. Following the 1847 Dee Bridge failure, Hawkshaw gave evidence on the application of iron to railway structures in 1849. He was responsible for some of the earliest lattice girder bridges in Britain, such as those over the Leeds and Liverpool Canal in Liverpool (1849).

In 1850 he moved to London, set up as a consulting engineer at 33 Great George Street, where he remained for the rest of his career. His practice was considerably broadened when in 1856 he became engineer for Holyhead harbour on the death of J M Rendel. On completion of these works in 1873 Hawkshaw was knighted. The resident engineer at Holyhead was Harrison Hayter, who joined Hawkshaw's staff as his chief assistant. From the mid-1850s the practice undertook all kinds of civil engineering work. This included sewerage schemes at Dover, Torquay, Lowestoft, Norwich, Ayr and the 7 mile Brighton intercepting sewer. In 1862, following failure of the St Germans sluice on the Middle Level Drainage, Hawkshaw advised the construction of thirteen large syphons. He became consultant to the drainage commissioners until his retirement. Land drainage and river works became an important part of his practice. He was consultant to the Witham drainage, and the Thames valley drainage commissioners. He reported on flood works at Lincoln and Norwich in 1877, and at Burton upon Trent in 1879. He recommended improvements to navigation channels on the Humber and Clyde and was consultant to the Weaver Navigation for many years.

Hawkshaw's practice was particularly noted for its dock and harbour works. One of his pupils was L. F. Vernon-Harcourt. Hawkshaw was consultant at Hull from 1862 until his retirement. Other works were carried out at Bristol, Boston, Penarth, Fleetwood, Maryport, Dover, Belfast, Greenock, Folkestone, Aberdeen, Alderney, and Wick. In London he was engineer for the south dock of the East and West India Dock Company.

In addition to his responsibilities for the Lancashire and Yorkshire Railway Hawkshaw was engineer for the Charing Cross Railway. He demolished Brunel's Hungerford suspension bridge and completed the Clifton suspension bridge as a tribute to Brunel, in partnership with W. H. Barlow. He was engineer for the South Eastern Railway from 1861–81, and for the Staines and Wokingham Railway, requiring a bridge across the Thames at Staines. He also designed Londonderry Bridge and South Bridge, Hull. Overseas, Hawkshaw was consultant to railways in Mauritius and Jamaica, and the Madras and East Bengal lines in India.

Hawkshaw's international reputation was not, however, reliant on colonial work. He was consultant on the Riga and Dünaberg, and Dünaberg and Vitebsk railways in Russia, and the Franz Josef Canal in Hungary. He was associated with some of the most important engineering projects of the second half of the nineteenth century. He was invited in July 1862 by Saʻid Pasha, viceroy of Egypt, to report on the proposed Suez Canal. His report induced Said to let the project proceed, and De Lesseps later acknowledged his debt to him. Hawkshaw attended the international congress on the proposed Panama Canal in 1879, but was critical of De Lesseps's plans. Hawkshaw himself was engineer with J. Dirks for the Amsterdam Ship Canal (1862–78), 16 miles long.

Hawkshaw's practice was extensive from the 1860s onwards. His son, John Clarke, joined the practice after leaving Cambridge in 1865. In 1870 he and Harrison Hayter became partners in the firm, all three later becoming presidents of ICE.

In the 1860s Hawkshaw became interested in the idea of a channel tunnel. By 1867, following investigations by the geologist Hartzinck Day in 1865, H. M. Brunel's marine survey in 1865–6, and deep borings into the chalk near Dover and Calais, Hawkshaw was convinced of the practicability of the scheme. From 1872 until 1886 he acted as joint engineer with James Brunlees to the Channel Tunnel Company.

Hawkshaw was responsible for other important tunnelling works including the completion of the Inner Circle Railway between Mansion House and Aldgate with Barry, and also for the

East London Railway, which involved the conversion of Sir Marc Isambard Brunel's Thames Tunnel to railway traffic, and a new tunnel beneath London docks. The contractor on both these railways was Thomas A. Walker (1828–1889).

The Severn railway tunnel was arguably Hawkshaw's greatest engineering achievement, the longest railway tunnel in the country, and the longest subaqueous tunnel in the world. The tunnel was originally designed by Charles Richardson, with Hawkshaw as consultant, but problems from underground springs were so great that he took charge in 1879. The tunnel was completed in 1886 after thirteen years work.

Hawkshaw's approach to engineering was essentially a practical one, and this is reflected in his reports and professional papers. He was heavily involved in the activities of ICE. He was President 1862–3 and was one of the most prolific contributors to its discussions. He encouraged his assistants and partners to give papers on the works of his practice. He was a member of the Royal Society (elected 1855), the Royal Society of Edinburgh, the Royal Geographical Society, and the Geological Society.

Hawkshaw was frequently consulted by the government. In 1860 he reported on competing schemes for the Dublin water supply. In 1861 he designed the foundations of the Spithead forts for the War Office. In 1868 he was one of five commissioners selected to report on the country's fortifications. In the same year he was appointed arbitrator to decide compensation to shareholders affected by nationalization of the telegraph companies. In 1874 he became the royal commissioner responsible for investigating the pollution of the Clyde. In 1880 he was appointed to the Board of Trade committee on wind pressure on railways structures, established in the wake of the Tay Bridge failure.

Hawkshaw was in his seventies when the Severn Tunnel was completed. He formally retired at the end of 1888, his business being continued by John Clarke Hawkshaw, Harrison Hayter, James Murray Dobson, and latterly his grandson Oliver. He died at Belgrave Mansions, Grosvenor Gardens, London, in June 1891.

## JONES, Sir Harry David (1791–1866)

Harry Jones was the youngest son of John Jones (1751–1806) and Mary Roberts born at Landguard Fort on 14 March 1791. He joined Royal Military Academy, Woolwich, April 1805, passed 6 month probation in Ordnance Survey, and was commissioned 2nd lieutenant, Royal Engineers, September 1808.

Jones's first station was Dover, where he was employed on the fortifications. Promoted first-lieutenant in June 1809, he embarked with Lord Chatham for the Scheldt, landed on the island of Walcheren and served at the capture of Flushing. Returned to England January 1810, he was sent to the Iberian Peninsula the following April where he took part in defence of Cadiz and the relief of Tarragona. He joined Wellington's army for the assault and capture of Badajoz (19 April 1812), and continued through the campaign of 1812–13. He was at the battle of Vitoria (June 1813) and was recommended for special promotion. At the siege of San Sebastian, Jones led the assault of 25 July 1813 and held the breach until all were killed, wounded or taken prisoner. Jones was severely wounded, and remained a prisoner until the castle surrendered in September 1813. He was promoted 2nd captain in November 1813 and was again wounded in December 1813.

In February 1814, Jones was sent on a special mission to New Orleans. In 1815 he joined Wellington's army after Waterloo, was at the capture of Paris, and commanded the engineers at Montmartre. He remained in France with the army of occupation. On return to England in 1818, he was stationed at Plymouth. In 1822 he obtained six months' leave and accompanied his brother John on an inspection of the Netherlands' fortresses. In 1823 he moved to Jersey, and in 1824 was appointed adjutant and field-work instructor at the Royal Engineer Establishment at Chatham. In 1824 he married Charlotte, second daughter of the Revd Thomas Hornsby, rector of Hoddesdon, Hertfordshire. In July 1825 he was promoted 1st captain. In 1826 he was sent to Malta, then to north Africa, to Constantinople to report on the Dardanelles and Bosphorus defences, returning overland to England. In July 1835 he was appointed commissioner for municipal boundaries in England, and in December 1835 as a member of the commission for improvement of navigation on the River Shannon. In February 1836 he was appointed first commissioner for fixing municipal boundaries in Ireland, and in October secretary to the Irish railway commission, which reported in 1838. In January 1837 he was employed on special service to the Admiralty. In April 1839 he was appointed commanding royal engineer at Jersey, but in November he was seconded and appointed to the Shannon commission. In September 1840 he was promoted lieutenant-colonel. His services in Ireland were so appreciated that when in 1842 he was offered an appointment at headquarters, he was, at the urgent request of the lords of the Treasury, retained in Ireland. In October 1845 was appointed chairman of the board of public works in Ireland.

He was a member of the relief committee under Sir John Burgoyne. In March 1850, he was appointed to command the Royal Engineers in north Britain. In May 1851 he was appointed director of the School of Military Engineering at Chatham. In 1853 he accompanied Lord Lucan to Paris on a mission from the queen to Napoleon III. In April 1854 he was again sent to Paris by Lord Raglan, to report on a new pontoon adopted by the French.

In July 1854 Jones became full colonel, and on the declaration of war with Russia he was appointed Brigadier-General. Sir Charles Napier decided to attack the Russian fortress at Bomarsund, Gulf of Bothnia. French troops had to be used in the amphibious operation. Initially liaison officer, Jones commanded British engineers, marines, and naval artillery ashore. The Russians surrendered, and the British demolished the fortifications. Jones received thanks of the queen for his Baltic services. In October he returned to England and resumed his Chatham duties. In December 1854 he was appointed major-general and ordered to Constantinople as commanding royal engineer there, but on arrival joined the army before Sevastopol. On 24 February he replaced Sir John Burgoyne as commanding royal engineer. He advised Lord Raglan and distinguished himself by his energy, daily visiting the trenches. He was severely wounded in the forehead and was mentioned in dispatches by Lord Raglan. In 1855 he was made KCB.

Soon after the fall of Sevastopol, Jones's wound necessitated his return to England. In April 1856 he was appointed governor of the Royal Military College, Sandhurst. He was unpopular with the cadets, and in October 1862 a mutiny occurred.

Before the Crimean War Sir John Burgoyne, other Royal Engineers officers, and Palmerston wanted improved coastal fortifications. By 1859, to counter the perceived threat from Napoleon III, Palmerston wanted a major coastal fortification programme as part of British rearmament and defence reconstruction. He encouraged and exploited the popular invasion ‘panic’ and secured the appointment in August 1859 of the royal commission on defences of the United Kingdom with Jones as its chairman. Its report (February 1860), assuming the possible absence of the fleet and rejecting coastal defence vessels and floating batteries, recommended a massive and expensive programme of fortification of naval bases. Although controversial, most of the works recommended were eventually built. In July 1860 Jones was promoted lieutenant-general, and in August that year he became a colonel-commandant, Royal Engineers. In 1861 he was appointed honorary colonel of the 4th battalion of Cheshire rifle volunteers, and was made a GCB.

Jones read papers (published in its Proceedings) to ICE on breakwaters (1842), a diving bell (1846), and a Shannon bridge. He wrote several articles in the Professional Papers of the Royal Engineers, and in 1859 he compiled the second volume of the official journal of the ‘siege of Sebastopol’. Jones died, while still governor at Sandhurst, in August 1866.

## MALLET, Robert (1810–1881)

Robert Mallet was born in Dublin in June 1810, to John and Thomasina Mallet. His father came from Devon in 1780 to join his uncle's brass and copper founding. Mallet entered Trinity College, Dublin, in December 1826, where he studied mathematics and science, graduating in 1830.

In 1831 Mallet became a partner in his father's works, the same year marrying Cordelia Watson. He took charge of the foundry, expanding it and securing large contracts for much railway plant, permanent way materials, and ironwork required in building railway lines in Ireland. He raised and strengthened the 133 ton roof of St George's Church in Dublin, for which in 1841 he was awarded a Walker premium by the Institution of Civil Engineers. In 1836 J. and R. Mallet erected a number of swivel bridges over the Shannon. Contracts for Guinness & Co., included a deep artesian well, construction of steam barrel-washing machinery, and large coolers. Mallet was elected an associate of ICE in 1839, transferring to member in 1842. He surveyed the River Dodder in 1841 and devised a scheme to provide a supply water to parts of Dublin, and to secure reliable summer supply to paper mills along the river. Between 1845 and 1848 he supplied and erected ironwork at many railway termini, as well as engine sheds, workshops, and a 200 ft span timber viaduct over the Nore in co. Kilkenny. His foundry supplied the castings for the first Fastnet Rock lighthouse (1848–9). Mallet invented the buckled plate, patented in 1852, used widely for flooring. During the Crimean War, he designed and had built two large siege mortars capable of firing 36” shells over 1 mile.

In 1860, Mallet closed the foundry and moved to London, where established as a consulting engineer. He edited the Practical Mechanic's Journal, 1865–9, wrote extensively for The Engineer, and gave evidence as a scientific witness in patent cases. In 1863 he reported on the Hibernia and other Ruhr collieries in Germany, and in 1864 was involved in an abortive scheme to connect Dublin's main line railway termini. He investigated the use of the Thames Tunnel by the East London Railway, and the possibility of damage to the Royal Observatory at Greenwich.

Mallet's investigations in physical geology were directed towards four main areas: glacial flowage (1837–45), geological dynamics (1835 onwards), seismology (1845 onwards), and vulcanology (1862 onwards). Between 1850 and 1858, he coined no less than eight terms with the prefix ‘seism-’, including seismology. His classic paper to the Royal Irish Academy in 1846 on earthquake dynamics is regarded as one of the foundations of modern seismology. He investigated the great Neapolitan earthquake of December 1857 and established the first principles of observational seismology. With his eldest son, John W. Mallet, he compiled a Catalogue of the World's Earthquakes (1852–4) and a Seismographic Map of the World (1857), both published by the British Association.

Mallet published at least eighty-five papers, on such diverse topics as the corrosion of iron, alloys of copper with tin and zinc, atmospheric railways, the application of water power, fouling of iron ships, earthquakes, and volcanoes. He was elected a fellow of the Royal Society in 1854. He received a Telford medal and premium from the Institution of Civil Engineers in 1859, and the Cunningham medal from the Royal Irish Academy in 1862. In the same year, the University of Dublin bestowed on him an honorary master of engineering degree, followed two years later by an honorary LLD. He was awarded the Wollaston medal of the Geological Society in 1877. Mallet died in London, in November 1881.

## MESSENT, Phillip John (1830-1897)

Phillip Messent was born in Dover on 7th December, 1830. He obtained his engineering training in the office of Messrs. Walker, Burges and Cooper. He made the preliminary survey and contract drawings for the Tyne Piers, and in April, 1855, he assumed charge of the work as their representative.

The North Pier was commenced in October 1855 by Mr. Lawton under a contract which expired in 1864. After that date the work was done by the Tyne Improvement Commissioners under the charge of Mr. Messent. The South Pier was commenced under the same conditions as the North Pier, in 1856, contracted to Mr. Lawton until 1864. The work was carried on under Messent's direction as engineer to the Tyne Improvement Commissioners, without a contractor.

In 1859, it was suggested to make the Tyne a national harbour of refuge. In addition to construction of the Tyne Piers Mr. Messent designed the lighthouses at their heads and at the entrance to the river. He also designed cranes to extend the masonry superstructure of the piers without staging, and a new concrete mixer which was very favourably reviewed, and said to give excellent results. In addition to his duties on the Tyne, Messent was engaged frequently as a witness before Parliamentary Committees and as arbitrator or consulting engineer in connection with important works, including the Manchester Ship Canal, Aberdeen Pier and Graving Dock, Swansea Harbour, Cardiff Docks, Port Talbot Docks, the Ribble Navigation, the Aire and Calder Navigation and the Lower Thames Navigation.

Messent was elected a Member of ICE in February 1861. He never gave a Paper of his own to the Proceedings, but frequently contributed to discussions on subjects relating to river, harbour and sea-works. He died in London in April 1897.

## PEARSON Weetman (1856-1927)

Born in 1856, Weetman Pearson was put in charge of a brickyard before age 18. He visited USA for 4 months in 1875. He inherited S Pearson & Son in 1884 from his grandfather and bought out his father. Moved home and firm to London.

Acquired substantial tunnelling experience (underwater) on Hudson River and Blackwall tunnels with (later Sir) Ernest Moir.

Tunnelling and drainage experience led him to major projects in Mexico, including drainage for Mexico City (including 26 mile of canal), ports (including Veracruz, Puerto Mexico, and Salina Cruz), and railways.  At essentially the same time as his construction of the extended harbour at Veracruz (5 lengths of breakwater totalling 3250ft, the outer one using 35t concrete blocks placed in brickwork pattern by Goliath cranes), he also won construction of the new outer harbour at Dover.

Young (1966) notes that Pearson bought a (local?) cement works before construction at Dover, and sold it afterwards for twice the purchase price.  At Dover, steam dredgers roughly levelled the chalk foundation with finer work by men in diving bells.  Temporary staging using 100ft x 2ft x 2ft, 10t Tasmanian blue-gum timbers used to form a hand-over-hand trestle.  Contract price was £3.3 million.

## RENNIE, John (1761–1821)

John Rennie (snr) was born in June 1761 at Phantassie, (near East Linton) youngest of nine children of James Rennie (d. 1766), a farmer and owner of a brewery, and his wife, Jean, née Rennie (1720–1783). John attended school at Prestonkirk. His early interest in machinery was nurtured by Andrew Meikle (1719–1811), inventor of the threshing machine and improver of the windmill, who lived on the estate. Rennie started to work for Meikle when he was twelve, getting a grounding in practical mechanics. For two years (1775–7) he attended Dunbar high school, where he was singled out for ‘amazing powers of genius’ in mathematics and experimental and natural philosophy.

Rennie set up as a millwright in 1779. Among his first jobs was building a mill for his brother to house one of Meikle's earliest threshing machines. He combined practical work with studies at Edinburgh University from 1780 to 1783. He gained broad scientific interest as well theoretical engineering concepts. In 1783 Rennie took a study tour in England on canals, bridges, and machinery. At Birmingham, he met James Watt who needed a millwright to extend the scope of his steam engine. Boulton & Watt offered Rennie a job looking after their London business and erecting the engines for the Albion Mills, their revolutionary flour mill at Blackfriars. Rennie moved to London, setting up a workshop near the mill. The millwork for the twenty sets of grinding stones was supplied by Wyatt, but substitution of iron gearing instead of the customary timber was probably Rennie's idea. Rennie opened Albion Mills to visitors when production began in 1786, despite the secretive Watt's disapproval. The building burned down in 1791, but by then Rennie's was supplying millwork for customers in France, Spain, and Portugal. He made moving machinery for mills, breweries, and factories, including machines for the new Boulton and Watt factory at Birmingham. Most of the equipment for the new Royal Mint at Tower Hill was Rennie's. He was ingenious in improving mechanical devices. A pioneer in applying steam power to pile-driving and dredging, he was among the first to make regular use of ball-bearings, improved the water-wheel and diving bell, experimented with stone pipes for water supply, and contributed to the evolution of the gantry crane. To meet the demand for his machines, he built a larger factory in 1810 at Holland Street, Southwark.

In 1790, he married Martha Ann Mackintosh (d. 1806). They had nine children, of whom George Rennie (1791–1866) and Sir John Rennie (1794–1874) carried on his work. Also in 1790 he was appointed surveyor to the Kennet and Avon Canal, after which design and consultancy in civil engineering took up most of his time. Along the Kennet and Avon (1794–1813), 57 miles long and with seventy-eight locks, many bridges, and several aqueducts and tunnels, He laid out the Rochdale and Lancaster canals, the Lune aqueduct, the Aberdeen and Crinan canals, the Royal Canal of Ireland, and the Royal Military Canal. Rennie also took on a multitude of river navigation and harbour improvements, fen drainage schemes, and waterworks. In London, he was key in expansion of the commercial docks (1800–05) and with Ralph Walker to the East India docks (1803–6), and West India docks (1809–17).

For the Admiralty, Rennie made improvements to Thames naval dockyards. His grandest work for government was Plymouth breakwater, started in 1811 and completed in 1848.

His thoroughness was key to his reputation and he was often asked to adjudicate on others' projects. A notable collaboration was with Robert Stevenson on the Bell Rock lighthouse off Arbroath (1807–10) although apportionment of responsibility led to prolonged disputes between their descendants.

Rennie is chiefly admired as a bridge-builder, and his crowning achievements were Waterloo Bridge, Southwark Bridge, and London Bridge, all constructed by Jolliffe and Banks.

Rennie was elected fellow of the Royal Society in 1798, but declined the knighthood offered to him by the prince regent when Waterloo Bridge was opened. He died in October 1821 in Southwark. He was buried in St Paul's Cathedral.

## RENNIE, Sir John (1794–1874)

John Rennie (jnr) was born in Blackfriars, London, in August 1794, the second son of John Rennie (1761–1821), civil engineer, and his wife, Martha, née Mackintosh. His elder brother was George Rennie, also a civil engineer. In 1809 he entered his father's office, where he acquired a practical knowledge of his profession. During the early stages of the building of Waterloo Bridge (1811–13) he worked with the resident engineer, James Hollingsworth. He produced working drawings for Southwark Bridge (1814–19) and selected blocks of Peterhead granite for the abutments. During his pupillage he also worked on the Kennet and Avon Canal and helped survey ports on the Tyne and Scottish coast.

In 1819–21 Rennie undertook an extensive grand tour which took in France, Switzerland, Italy, Greece, the Turkish coast, and Egypt, visiting antiquities, quarries, and engineering works. He returned to find his father dying, and with his brother George took over the firm, specializing in civil engineering. The most important of his undertakings was the construction of London Bridge, the designs of which had been prepared by his father. After many controversies, the bridge was opened in 1831, when Rennie was knighted, one of the first professional engineers to be thus distinguished.

As engineer to the Admiralty, a post in which he succeeded his father up to 1831, Rennie carried on various works at Sheerness, Woolwich, Plymouth, Portsmouth, and Ramsgate. At Plymouth he completed his father's great breakwater. Sir John Rennie was primarily a hydraulics engineer, and much of his career was spent in modifying commercial harbours and docks, including Whitehaven and Cardiff. Overseas, he built the Ponte Delgada breakwater in the Azores. He completed drainage works in the Lincolnshire fens commenced by his father and, in conjunction with Telford, constructed the Nene outfall near Wisbech (1826–31). The J. and G. Rennie shipyard established in the 1830s at Greenwich, built vessels of widely varying type, including screw ships incorporating engines made at their ironworks near Blackfriars Bridge. Among Sir John's personal contributions to this side of the business was the design of fixed floats for paddle wheels.

Rennie was elected a fellow of the Royal Society in 1823 and of the Zoological Society in 1825. He joined ICE only in 1844 but was made President in 1845–8, reviewing the (recent) history of civil engineering in his inaugural address in 1846. His contributions to ICE Proceedings include an analysis of Ostia harbour; an Account of Plymouth Breakwater (1848), a magnum opus entitled Theory, Formation and Construction of British and Foreign Harbours (1851–4). Rennie was a connecting link between older engineers such as Brindley, Smeaton, Telford, and his father, John Rennie, and younger men such as the Stevensons and the Brunels. He was more amiable and sociable than his father or brother and enjoyed the comfort and dignity which his family's professional success afforded him. He retired about 1863 and died near Hertford, in September 1874, just after his 80th birthday.

## SCOTT-RUSSELL, John (1808-1882)

John Scott-Russell was born in May 1808 in Parkhead, Glasgow, the son of Reverend David Russell and Agnes Clark Scott. He spent one year at St. Andrews University then transferred to Glasgow University where he graduated in 1825, age 17. He moved to Edinburgh where he taught mathematics and science at the Leith Mechanics' Institute. In 1832 he was elected professor of Natural Philosophy at Edinburgh University, pending election of a permanent professor. While in Edinburgh he experimented with steam engines, using a square boiler from which the Scottish Steam Carriage Company produced a steam carriage with two cylinders developing 12 horsepower each. Six were constructed in 1834. The road trustees objected that it wore out the road and placed various obstructions in the road. Two coaches were sent to London where they ran between London and Greenwich.

In 1834, while conducting experiments to determine the most efficient design for canal boats, he identified the solitary wave, also described as a wave of translation. He was appointed in 1836 by the British Association to investigate the whole subject of waves. His report records observations on waves of the sea, tidal-waves, and on the effect of estuaries in modifying tides, waves generated in canals and other confined channels. He built wave tanks at his home, but his experimental work gave results at odds with Newton's and Bernoulli's theories, so Airy and Stokes had difficulty accepting his experimental observations. In 1876 however, Lord Rayleigh published a theory supporting Scott-Russell's experimental observation.

Many of Scott-Russell's early experiments were conducted under the British Association to which he contributed throughout his life. In 1844, Scott-Russell moved to London, formalising his activities as a ship-builder at Milwall. He contributed an article on the Steam engine and steam navigation for Encyclopaedia Britannica. He was secretary of the Royal Society of Arts committee to organise a national exhibition, with two subsequent exhibitions. Scott- Russell became RSA's secretary for the 1851 Great Exhibition.

He became a member of ICE in 1847, attending regularly and making frequent contributions, was elected to Council in 1857 and became a vice-president in 1862. He was elected a Fellow of the Royal Society in 1849. In 1860 the Institution of Naval Architects was set up at a meeting at his house in Sydenham. He attended most meetings and in 1864 he published a 3-volume treatise on The Modern System of Naval Architecture which laid out the profiles of many of the new ships being built.

He died in June 1882 in Ventnor, Isle of Wight.

## STEVENSON, Robert (1772-1850)

Robert Stevenson was born in Glasgow; his father was a partner in a West India trading house but died of fever when Robert was an infant, leaving his widow, Jane Lillie, in straightened financial circumstances. Robert was educated as an infant at a charity school. His mother intended Robert for the ministry. But in 1787, his mother married Thomas Smith a tinsmith, lamp-maker and ingenious mechanic who had been appointed engineer to the Northern Lighthouse Board.

Robert served as Smith's assistant, and at 19 was entrusted with construction supervision of a lighthouse on Little Cumbrae in the river Clyde. He applied himself to surveying and architectural drawing and attended lectures in mathematics and physical sciences at the Andersonian Institute at Glasgow. Study was interleaved with work - his next project was lighthouses on Orkney. In winter months he attended lectures in philosophy, mathematics, chemistry and natural history, as well as moral philosophy, logic and agriculture at the University of Edinburgh. He did not take a degree, however, having a poor knowledge of Latin, and none of Greek.

In 1797 Robert Stevenson was appointed engineer to the Lighthouse Board in succession to Smith. He married Smith's eldest daughter Jean (his stepsister) in 1799, and in 1800 he became Smith's business partner.

It is probable that his most important work was the Bell Rock Lighthouse, long in gestation and long and hazardous in construction. The involvement of John Rennie (the elder) led to contention for the credit, particularly between Alan Stevenson, Robert's son, and John Rennie, son of the consulting engineer. The Northern Lighthouse Board, give full credit to Stevenson.

Stevenson served for nearly fifty years as engineer to the lighthouse board, until 1843, during which he designed and supervised construction and improvement of numerous lighthouses. He innovated in the choice of light sources, mountings, reflector design, the use of Fresnel lenses, and in rotation and shuttering systems providing lighthouses with individual signatures allowing them to be identified by seafarers, for which he was awarded a gold medal by William I of the Netherlands.

The period after Waterloo and the end of the continental wars was a time of much improvement of the fabric of the country, and engineering skills were much in demand. Besides his work for the Northern Lighthouse Board, he acted as a consulting engineer on many occasions, working with many leading engineers. Projects included roads, bridges, harbours, canals and railways, and river navigation. He invented movable jib and balance cranes for lighthouse construction.

He was made a Fellow of the Royal Society in 1815. He published an Account of the Bell Rock Lighthouse in 1824; a paper establishing that the North Sea was eroding the eastern coastline of the United Kingdom. He contributed to the Encyclopedia Britannica and the Edinburgh Encyclopedia, and published in a number of the scientific journals of the day. In 1828 he became a member of the Institution of Civil Engineers.

Three of Stevenson's sons became engineers: Alan who became a partner in the firm about 1832, David who became a partner in 1838, and Thomas, who became a partner in 1846 on his father's retirement. He also had a daughter, who assisted in writing and illustrating an account of the Bell Rock Lighthouse construction. He died in Edinburgh in July 1850.

## STEVENSON, Alan (1807-1865)

Alan Stevenson was born in Edinburgh the eldest son Jean Smith and her husband and step-brother Robert Stevenson. He was brother of David and Thomas Stevenson. Alan was educated at the High School in Edinburgh, and attended University of Edinburgh to study Latin, Greek and mathematics with a view to becoming a member of the clergy. In 1823 however, he began a four-year apprenticeship at his father's business. He continued to study part-time at the University and graduated with an MA in 1926.

Between 1843 and 1853 he built 13 lighthouses in and around Scotland. Among his notable works is the Skerryvore Lighthouse. He was Engineer in Chief to the Northern Lighthouse Board from 1843 to 1853.

In 1838 he was elected a Fellow of the Royal Society of Edinburgh his proposer being James David Forbes. In 1840 the University of Glasgow conferred on him an honorary LLB degree. He died in Portobello in December 1865.

## STEVENSON, David (1815-1886)

David Stevenson was born in January 1815 at 2 Baxters Place at the top of Leith Walk, Edinburgh, the son of Jean Smith and engineer Robert Stevenson. He was brother of the lighthouse engineers Alan and Thomas Stevenson. He was educated at the High School in Edinburgh then studied at the University of Edinburgh. In 1838 he became a partner in his father's (and uncle's) firm of R & A Stevenson.

In 1844 he was elected a Fellow of the Royal Society of Edinburgh. In 1853 he moved to the Northern Lighthouse Board. Between 1854 and 1880 he designed many lighthouses, all with his brother Thomas. He also helped Richard Henry Brunton design lighthouses for Japan, inventing a novel method for allowing them to withstand earthquakes. His sons David Alan Stevenson and Charles Alexander Stevenson continued his work after his death, building nearly thirty further lighthouses.

Non-lighthouse engineering included the Edinburgh and Leith Sewerage Scheme and the widening of North Bridge in Edinburgh.

In 1868/9 he served as President of the Royal Scottish Society of the Arts. He died in North Berwick in July 1886.

## STEVENSON, Thomas (1818-1887)

Thomas Stevenson was born in Edinburgh in July 1818, the fifth son of the well-known Lighthouse Engineer, Robert Stevenson. Thomas joined his father's office at 17, and after serving his pupilage and superintending various works, he was taken into partnership by his brothers Alan and David in 1846.

His firm was chiefly engaged in the construction of harbour, dock, river, and lighthouse works in Scotland, but they were also called upon to design improvement works for many rivers and harbours in England and Ireland.

In 1855 Thomas Stevenson and his brother David were appointed Engineers to the Northern Lighthouse Board. Their firm designed and erected 28 lighthouses, some particularly difficult.

Being the junior partner of the firm facilitated his investigations on harbour construction, with particular attention to the forces to be overcome in works exposed to heavy seas and/or in deep water. He devoted special attention to ascertaining wave heights, laws of wave propagation, and their action on man-made structures. He measured wave forces by instruments that he had devised. Results of his wave-observations, and the laws deduced from them, were given in papers to the Royal Society of Edinburgh, and to the Edinburgh Philosophical Journal.

In 1852, he described how wave heights increase with fetch; and developed formulae to estimate wave diffraction into harbours, of considerable value to marine engineers. Despite this ground-breaking work, he always considered it as an approximation, and was clear that more was needed.

Having originally written an article on 'Harbours' for the 'Encyclopaedia Britannica,' he then developed this further as a separate treatise on 'The design and construction of Harbours,' Stevenson (1864, 1874).

Thomas devoted much attention to lighthouse optics, and devised many important improvements, in which he was assisted by Alan Brebner. He was awarded gold medals by the Royal Scottish Society of Arts and at the French Exhibition of 1878, his work in lighthouse optics were further awarded a gold medal.

He was an early supporter of the Scottish Meteorological Society, and took an active part in the establishment of the high-level Observatory at Ben Nevis.

Mr. Stevenson was a devoted member of the Church of Scotland, writing several pamphlets on religious questions, one of which was reprinted for students. He was elected Fellow of the Royal Society of Edinburgh in 1845, acted as a member of Council, as one of its Vice-Presidents, and in 1885 was elected its President. He was elected Member of ICE in February 1864.

## STONEY, Bindon Blood (1828–1909)

Bindon Stoney was born at Oakley Park in King's county, Ireland, June 1828, to George Stoney and Anne Blood. At Trinity College, Dublin he studied civil engineering, graduating with distinction in 1850. He started work as assistant to the 3rd Earl of Rosse in the observatory at Birr Castle. Using a 72 in. reflecting telescope (the largest in the world at the time), he mapped the spiral form of the Andromeda nebula, work which led directly to the conclusion that such galaxies were as numerous as the stars in our own galaxy.

Stoney worked on railway surveys in Spain in 1852–3 before joining James Barton as resident engineer constructing the Boyne Viaduct at Drogheda, probably the earliest use of metal girders of considerable span in which latticed bars were substituted for a continuous plate web, and the cross-sections of the web members as well as of the flanges were proportioned to stresses imposed by the rolling load. Stoney contributed to the final design of the ironwork and the method of erection. The experience gained led him to a thorough study of stresses and strains in girders, resulting in publication in 1866 of a treatise in two volumes, The Theory of Strains in Girders and Similar Structures.

In 1856 Stoney was appointed assistant engineer to the port authority of Dublin, at that time the Ballast Board. In 1862, he succeeded George Halpin as chief engineer, holding that post until his retirement in 1898. Stoney developed an original method of placing large monolithic blocks of masonry concrete, up to 350 tons in weight, for port construction, and promoted the large-scale use of cement. Under his direction, half the quay walls along the River Liffey were reconstructed to form deep-water berths. He constructed the North Wall extension which forms part of the Alexandra basin. Shortly after the project was completed in 1881, he received an honorary LLD from the University of Dublin and was also elected a fellow of the Royal Society. He deepened the channel between Dublin Bay and the city quays, designing powerful dredging plant and hopper barges of unprecedented capacity. He also supervised the rebuilding of Grattan (1874) and O'Connell (1880) bridges across the River Liffey, as well as designing the Beresford swing bridge near the custom house.

In 1879 Stoney married Susannah Frances, daughter of John Francis Walker QC of Grangemore, co. Dublin. He was elected an associate of ICE in 1858, became a full member in 1863, and served as a member of Council from 1896 to 1898. He was elected a member of the Institution of Civil Engineers of Ireland in 1857, serving as joint honorary secretary (1862–70), and as president in 1871 and 1872. He was also a member of the Royal Irish Academy and of the Royal Dublin Society, and a fellow of the Institution of Naval Architects. The ICE awarded him a Telford medal and premium in 1874 for his paper, ‘The construction of harbour and marine works with artificial blocks of large size’. He contributed eight papers to the Institution of Civil Engineers of Ireland between 1858 and 1903, including his presidential address (1872) and a paper on strength and proportions of riveted joints which was also published in book form (1885). He also contributed four papers dealing with the theory of structures to the literature of the Royal Irish Academy. Stoney died in Dublin, in May 1909.

## VERNON-HARCOURT, Leveson Francis (1839–1907)

Francis Vernon-Harcourt was born in London in January 1839, the second son of Admiral Frederick Edward Vernon-Harcourt (1790–1883) and Marcia (1803/4–1868), daughter of Admiral Richard Delap Tollemache, formerly Halliday. Educated at Harrow School and at Balliol College, Oxford, he obtained a first class in mathematics in 1861, and graduated with a first class in natural science in 1862.

From 1862 to 1865 Vernon-Harcourt was a pupil of John Hawkshaw, and was employed on the Penarth and Hull docks. After serving as an assistant, he was appointed in November 1866 as resident engineer on the East and West India docks until their completion in January 1870. He married Alice, younger daughter of Lieutenant-Colonel Henry Rowland Brandreth RE, with whom he had a son and two daughters. He took the position as Resident Engineer at Alderney harbour. His paper on this subject was the first of many to ICE. From 1872 to 1874 Vernon-Harcourt was based in Ireland and was resident engineer on the Rosslare harbour works and the railway to Wexford. He returned to London, and between 1875 and 1878 carried out surveys for Hawkshaw on the River Clyde, the River Witham and the upper reaches of the River Thames.

He started as a consulting engineer in 1882 in Westminster, and was elected professor of civil engineering at University College, London. He devoted himself to harbours and docks, rivers and canals, and water supply. He wrote numerous books. He was elected associate in December 1865, and full membership in December 1871. He was awarded the Telford and George Stevenson medals, six Telford premiums, and a Manby premium. His papers covered many aspects of his field. He contributed articles to the Royal Society, the Society of Arts, and the British Association, as well as to the Encyclopaedia Britannica (9th edn) and other publications. He retired in 1905, being elected emeritus professor in 1906.

Vernon-Harcourt, who was fluent in French, regularly contributed to navigation congresses. He represented ICE in Brussels (1898), Paris (1900), Düsseldorf (1902), and was a government delegate to the Milan Congress in 1905. He was the British member of the jury for the Paris Exhibition of 1900, and again at the St Louis Exhibition of 1904. In 1906 he was appointed to the international commission of the Suez Canal. He was the oldest member of the council of PIANC when died in September 1907.

## VETCH, James (1789-1869)

James Vetch was born at Haddington, East Lothian, in May, 1789, the third son of Robert Vetch, of Caponflat, East Lothian.

He was educated at Haddington and Edinburgh. In 1804, having been nominated by Lord Chatham, he joined the Cadet College at Great Marlow, transferring in 1805 to the Royal Military Academy at Woolwich. He left Woolwich in 1806, to join the Trigonometrical Survey at Oakingham, as Assistant-Engineer under Mr. Robert Damson, with whom he remained till the summer of 1807.

He was commissioned as 2nd Lieutenant in the Royal Engineers in July, 1807; was promoted in 1808 to first Lieutenant; and in July, 1813, to Captain.

After serving for three years at Chatham and Plymouth, he was ordered in 1810 to the Iberian Peninsula, to join the blockade of Cadiz until it was raised in 1812. He stayed in the south of Spain until his return to England in 1814.

During the next six years, Captain Vetch commanded a company of sappers and miners, and was stationed first at Spike Island, in Cork Harbour, and afterwards at Chatham where he devoted himself to the study of geology and other scientific pursuits. While constructing the fort on Spike Island, he needed to remove an old tower, with 8 feet thick walls. Having paid considerable attention to military mining, he successfully blew it up bodily.

In 1821, Duke of Wellington, appointed Captain Vetch to the Ordnance Survey where he conducted the triangulation of Orkney and Shetland Islands, and the Western Islands. In the winter months, Captain Vetch attended lectures at Edinburgh University.

In 1822 Captain Vetch submitted to the authorities an invention for throwing a line from the shore to a vessel in distress.

In 1824, he obtained permission to retire on half-pay, remaining in Mexico till 1835; developing the Real del Monte, Bolanos, United Mexican, and other mining concerns. By laying-out and constructing good roads, and organizing efficient systems of transport, he paved the way for the more extended mining operations.

In 1836 Captain Vetch was appointed one of the Commissioners for settling the Irish borough boundaries; and, on completing this, he was employed by the Birmingham and Gloucester Railway Company as Resident Engineer. He was then engaged in Ireland and Scotland on the reclamation of tide-lands and formation of embankments. He was frequently consulted by the Commissioners of Woods and Forests, and by the Admiralty. In 1843, he published an 'Enquiry into the means of establishing ship navigation between the Mediterranean and Red Seas,' which attracted considerable attention.

During 1844-46 Captain Vetch was requested by the Tidal Harbours, and Harbours of Refuge Commissions to explore any advantages by using wrought-iron framework for piers or breakwaters. In 1845, he was asked by Sir Byam Martin to review designs for a Harbour of Refuge at Dover submitted by: Sir John Rennie, Messrs. Walker, Cubitt, Vignoles, George Rennie, James M. Rendel, the late Sir Harry Jones, and Sir William Denison..

In 1846, Captain Vetch was appointed Consulting Engineer to the Board of Admiralty on questions relating to railways, bridges, harbours, rivers, and navigable waters of the United Kingdom. In 1847 this appointment was abolished, and Captain Vetch was appointed to the new Harbour Conservancy Board at the Admiralty.

In 1859 he was a member of the Royal Commission on Harbours of Refuge chaired by Admiral Sir James Hope. Over 16 years, Captain Vetch was employed at the Admiralty having to judge the plans of Civil Engineers of eminence and reputation. This may be gathered from a letter by Admiral Washington, the late Hydrographer, in 1858: " (Captain Vetch) had at once thrust upon him, in the very first year of office at the Admiralty, one hundred harbour and railway bills, on which he was required to report to Parliament, and the work was such that, even Sir Francis Beaufort, with all his experience, shrank from it, and would have resigned his post had he not been relieved of it." Captain Vetch retired from the Admiralty in 1863, when his office was abolished, and the duties transferred to the Board of Trade.

He was elected a Fellow of the Geological Society in 1818; of the Royal Geographical Society and the Royal Society in 1830; and an Associate of ICE in March, 1839. He died at the age of eighty in December 1869.

## WALKER James (1781-1862)

James Walker was born at Falkirk, in 1781. His education commenced at Falkirk school, and continued at University of Glasgow. He had talent and was industrious; so that his studies were completed with distinction in a comparatively short time; and in 1800, when he was nineteen, he was articled to his uncle, Ralph Walker, at that time an Engineer in practice in London. He was employed at the East and the West India Docks, and became conversant with the best class of constructions, eventually succeeding his uncle in the greater portion of his works.

He was appointed Engineer to the Commercial Docks in 1806, where extensions and improvements to the time of his death were designed by, and executed under, Mr. Walker and his partner, Mr. Burgess.

At the decease of Mr. Telford in 1834, many of his important works were taken on by Mr. Walker. For many years after, he was frequently consulted by branches of Government, and many public boards and corporations. He made numerous reports upon the City of London sewers, and navigation of the River Thames. He had under his charge the bridges at Blackfriars and Westminster, executing extensive works for their maintenance.

Any attempt to mention all the works upon which Mr. Walker had been engaged at some period of his long professional career, would involve the enumeration of nearly all the principal undertakings of the kingdom; but among them must be mentioned construction of Vauxhall Bridge over the Thames, and Victoria Bridge over the Clyde; the great repairs of the Caledonian Canal and the Crinan Canal; the Coffer Dam and River Wall of the New Houses of Parliament; the extensive works of the Birmingham Canal, including the Tame Valley Canal, the Betley Canal, and the Netherton Tunnel; extensions of the Bute Docks at Cardiff; the Pier and Harbour of Granton; improvements of the Harbour of Belfast, and Harbour Works at Dover; the designs and execution of the Harbours of Refuge at Alderney, St. Catherine's, Jersey, Dover, and Harwich, on the former of which he was penning a Report in his own hand on the day before his decease; the Tyne Piers; the completion of Plymouth Breakwater, and the foundation for the Fort about to be constructed there for the War Department.

But, perhaps, the most lasting monuments of his skill may be found in the various Lighthouse Works of the Corporation of the Trinity House. As a difficult and successful engineering work, the Bishop Rock Lighthouse, off the Scilly Islands, may be placed in the foremost rank with those of John Smeaton with the Eddystone Lighthouse; of the elder Stevenson, with the Bell Rock Lighthouse; and of that built on the Skerryvore, by Alan Stevenson, his son.

The reports from Mr. Walker's pen are very numerous giving the history of many important works. He was a strict disciplinarian, was very regular in his business habits, and indefatigable in his attention to the works under his charge. He was fortunate in attracting to him a number of men who have since risen to eminence in the profession; and he was very ably seconded by his partner, Mr. Burges, and subsequently by the late James Cooper.

Mr. Walker was a fellow of the Royal Societies of London and of Edinburgh, a member of the Senate of the University of London, and an honorary member of several foreign societies.

He joined ICE as a Member in 1823, was elected a Member of the Council and Vice-President in 1826, succeeding the first President, Mr. Telford, in 1834, and occupied the chair for nearly eleven years. Under his presidency, the Institution made important strides; the number of the Members increased; the Society was enabled to move to 25, Great George Street, Westminster. Mr. Walker had a peculiar talent for guiding the Meetings, and for eliciting information from Members and Visitors. His extensive acquaintance among those interested in Engineering science persuaded many distinguished visitors to attend the Meetings, and, in some instances, to join the Society.

Mr. Walker continued to exercise his profession up to the period of his decease, and to the very last he retained his vigorous and shrewd intellect. He was eminently a man of business, and he had, at least, as much skill 'in the engineering of men as of matter.' He died in October 1862.

## WHITE. Martin, 1779-1865

Martin White was born at Hayling Island (probably) in 1779, first going to sea in the navy in July 1794, and qualifying as a Lieutenant in December 1800. His first command was (probably) the Pigmy which helped blockade Le Havre, St Malo and Granville, during which he 'surveyed at the same time the approaches to each'. He was promoted to Commander and given command of the Vulture, Guard Ship on the Jersey Station, from which "every opportunity was taken … to obtain correct soundings". This activity continued with commands of Fox and Shamrock. A Survey of the Channel Island and the Coast of France was submitted to the Hydrographic Office in 1815, and his Sailing Directions for the English Channel published in 1834 became the definitive guide. His surveying abilities were held in high regard, being especially commended by Captain Thomas Hurd, Hydrographer to the Navy; and Admiral Sir George Cockburn, the senior Naval Lord. Martin White retired from active service in October 1846 having surveyed some 60,000 square miles of ocean. He was promoted to Admiral (Retd) in 1862 and died in June 1865.

## ICE Presidents, 1820-1910

| Dates | President |
|---|---|
| 1820-34 | Thomas Telford |
| 1835-45 | James Walker |
| 1845-48 | Joshua Field |
| 1848-50 | Sir John Rennie |
| 1850-52 | Sir William Cubitt |
| 1852-54 | James Meadows Rendel |
| 1854-56 | James Simpson |
| 1856-57 | Robert Stevenson |
| 1857-59 | Joseph Locke |
| 1859-62 | George Parker Bidder |
| 1862-64 | John Hawkshaw |
| 1864-66 | John Robinson McClean |
| 1866-68 | Sir John Fowler |
| 1868-70 | Charles Hutton Gregory |
| 1870-72 | Charles Blacker Vignoles |
| 1872-74 | Thomas Hawksley |
| 1874-76 | Thomas E. Harrison |
| 1876-78 | George Robert Stevenson |
| 1878-80 | John Frederick La Trobe Bateman |
| 1880-81 | William Henry Barlow |
| 1881-82 | James Abernethy |

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|---|---|
| 1882-83 | William Armstrong |
| 1883-84 | James Brunlees |
| 1884-85 | Joseph Bazalgette |
| 1885-86 | Frederick Joseph Bramwell |
| 1886-87 | Edward Woods |
| 1887-89 | George Barclay Bruce |
| 1889-91 | Sir John Coode |
| 1891-92 | George Barkley |
| 1892-93 | Harrison Hayter |
| 1893-94 | Alfred Giles |
| 1894-95 | Sir Robert Rawlinson |
| 1895-96 | Sir Benjamin Baker |
| 1896-98 | Sir John Wolfe-Barry |
| 1898-99 | Sir William Henry Preece |
| 1899-90 | Sir Douglas Fox |
| 1900-01 | James Mansergh |
| 1901-02 | Charles Hawksley |
| 1902-03 | John Clarke Hawkshaw |
| 1903-04 | Sir William Henry White |
| 1904 | Sir Guilford Lindsey Molesworth |
| 1905-06 | Sir Alexander Binnie |
| 1906-07 | Sir Alexander Kennedy |
| 1907-08 | Sir William Matthews |
| 1908-10 | Sir James Charles Inglis |
| 1910 | Sir Alexander Siemens |