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## Historical experience of vertical breakwaters in the United Kingdom

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Reproduced from a paper presented at:  
Coasts, Marine Structures and Breakwaters 2009 Conference  
EICC, Scotland  
16-18 September 2009





# ***HISTORICAL EXPERIENCE OF VERTICAL BREAKWATERS IN THE UNITED KINGDOM***

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## **1. Introduction**

### **Background**

This paper discusses the evolution and construction of vertical breakwaters around UK, particularly those constructed during the major period of harbour construction (1830 – 1900). Many of those breakwaters survive, and their stability is important to continuing operation of the harbours protected. The most common form of construction until relatively recently used blockwork walls (generally stone blocks) founded on rubble mounds, so this paper concentrates particularly on that form of construction. The design and stability of Alderney Breakwater had previously been described by Allsop et al (1991). Significant new research has improved understanding and prediction of impulsive wave loads, and the responses of blockwork walls to wave loadings.

### **Outline of this paper**

This paper revises and extends an initial discussion by Allsop & Bray (1994) for a workshop in Japan. The design, form, use, and construction of the breakwater types discussed here are closely inter-linked, and are strongly influenced by the exposure of the particular site; by availability and quality of local materials; by constructional techniques and equipment available locally at the time; and by the experience of the designer and/or builder. A simplified history of the design / construction of UK breakwaters is therefore presented in section 2 to identify the overall context within which particular structures were constructed. The forms of construction are described in more detail in section 3. A brief insight into an early use of caissons is given in section 4. The main failure modes for these types of structures; and example methods to analyse or to design against them are discussed in section 5. Examples of deterioration failures are covered in section 6, including a few comments on Alderney Breakwater. Summary guidance is summarised in section 7.

## **2. Historical perspective**

Around the shores of the Mediterranean, ancient breakwaters had been constructed of stone blocks, sometimes with concrete or cementitious infill, from around 2000 years ago. 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 have been dated to Roman times. In general, Roman ports in the UK were developed in estuaries, particularly the Thames and Medway, and used embankments and quay walls, primarily of

timber. Few details of construction of breakwaters or coastal walls are recorded before the late 1600's, and much of the information now available dates from breakwaters constructed in the late 1700 and through the 1800s. For instance, very few details are available on the construction of the Cobb at Lyme Regis, attributed variously to the 12<sup>th</sup>, 13<sup>th</sup> and 16<sup>th</sup> centuries. At Dublin, work began in 1716 on a Mole to protect the south side of the channel from Ringsend to Poolbeg. The South bank provided only limited protection for shipping and in 1753, so was replaced by the Great South Wall out to Poolbeg Lighthouse, lit for the first time in September 1767. The breakwater at Eyemouth used stone blocks on inclined planes, 1767, and a breakwater at Brixham was started in 1799. One notably early example for which more information is available is the construction in 1663-1678 by British engineers of the Greate Mole at Tangier in north-west Africa described by Routh (1912), discussed in section 4.

A major expansion of ports between about 1820 and 1900 saw vertical breakwaters or piers being constructed at Wick (see Paxton, 2009), Brixham (extended in 1837), Alderney started in 1846, Dover 1847, Tynmouth 1855, Aberdeen 1873, Holyhead 1876, Fraserburgh 1877.

The original purpose of the larger harbours around the UK was military, with naval requirements setting the position, orientation and plan for harbours at Dover, Portland, Plymouth, Holyhead, St Catherine's and Alderney. Some were never completed, and most have abandoned naval use, and now support commercial, fishing or leisure activities. Other coastal harbours (usually much smaller) were built for fishing fleets, or later for trading vessels. A few harbours were constructed as "harbours of refuge" (including some of the above), to be used by fishing boats and trading vessels during storms. These larger, harbours were easier to enter than small coastal fishing harbours where reflective walls and narrow entrances may still cause potentially

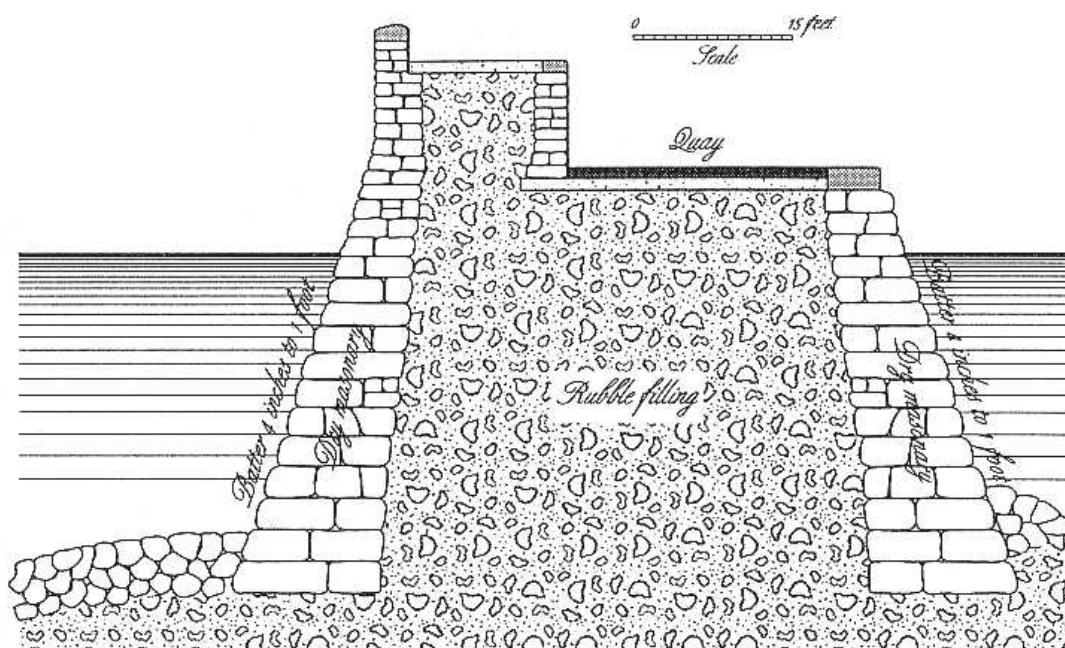
dangerous conditions close to the harbour entrance.

### 3. General forms of historic vertical breakwaters

The more common types of breakwater or seawall in harbour or coastal works around the UK were walls of simple vertical or battered slope formed of stone or concrete blocks, founded on a mound of quarry rock. Such structures were relatively cheap to construct when labour costs were low, and used a minimum of material. Breakwater walls are usually double-sided, but seldom of blockwork all the way through. Quays or seawalls are usually backed by natural or imported materials. Breakwaters (also known as *Moles* or *Piers*) were often used as quays for cargo handling.

#### Breakwater configurations

An example breakwater section in Figure 1 from St Catherine's harbour on Jersey (see Bray & Tatham, 1992, Hold, 2009), constructed at about 1855-60 shows the dry masonry walls, the rubble filling between the walls, and the rubble mound on which the walls are founded. The figure does not show low water, which is slightly above the wall foundation level shown, see also Figure 3. For all of these breakwaters, the depth at which foundation blocks could be placed was set almost entirely by diving capabilities at the time. Rubble material placed below lowest tide level was trimmed by divers to accept the foundation stones. Divers also guided placement of the foundation blocks. In greater water depths, the rubble mound foundation became a significant proportion of the overall structure height. Thus many of the deep water breakwaters around the UK have been vertically-composite with substantial mounds. Advances in diving technologies over the period 1850-1880 significantly increased those depths, reducing the proportional height of the rubble mound, so that the mound represents a smaller proportion of the overall structure height.



**Figure 1** St Catherine's breakwater, 1856 (after Bray & Tatham)

### Blockwork

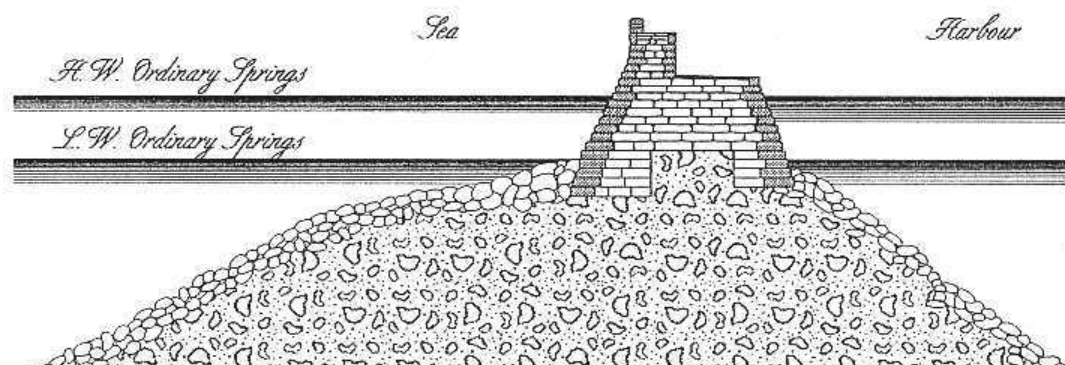
Quarried stone is not naturally available in the rectangular shapes needed to form a coherent and stable wall. Production and dressing of stone blocks to acceptable sizes and tolerances used to be a routine task in civil engineering, but has become significantly less economic as labour costs have risen. The use of concrete to form blocks might have been started by the Romans, but disappeared from construction practice for coastal structures until commercial production of Portland cement around 1845. Many breakwaters before about 1850 therefore used large stone blocks to form the outer skin of the wall, with the core formed from smaller blocks and masonry off-cuts and/or rubble infill, or even sand. An unusual exception is the Stone Pier at Margate where the core is formed by large chalk blocks.

Hewn stone, often granite, was laid in bond, generally at a slight batter off vertical. Blocks were generally laid dry or in lime or pozzolana mortar up to about 1900. Concrete filling was rarely used, and cement mortars became widely available only after about 1900, although lime and other mortars were used at least from 1650.

Concrete rather than stone blocks started to be more widely used after 1845.

Various methods have been developed to assist transfer tensile, bending, or shear loads between adjoining blocks, or between columns or courses of blockwork. These include the use of iron or steel cramps set in lead or mortar, stone or concrete keys, or joggle bag joints between blocks.

The use of concrete for filling breakwater walls, and/or to form the facing started to be used occasionally again after about 1830, becoming more prevalent after about 1870. There is no record of concrete being used for the North Pier at Eyemouth, 1767; Old Pier at Wick, 1823; the piers at Hynish, 1843, Buckie, 1855, and West Hartlepool, 1858. Pre-cast concrete blocks were however used at North Tyne in 1855; for Dover breakwater, 1866; and at Cork in 1877. Concrete filled bags formed a foundation to Fraserburgh breakwater in 1877, and for the Winton Pier, Ardrrossan in 1892. Concrete filling was used for the later stages of Alderney breakwater 1849-1866, the South Breakwater at Aberdeen, 1873; for the North Pier at Aberdeen, and the Fraserburgh breakwater, both in 1877.



**Figure 2 Alderney breakwater, 1851-64, (after Bray & Tatham)**

The sections of St Catherine and Alderney Breakwaters in Figures 1 and 2 are relatively typical of the larger breakwaters constructed between 1850 and 1880. Of these, Alderney is exposed to substantially more severe wave conditions, has suffered significant damage, provides us with more information on failure modes and responses, and is therefore given more attention later.

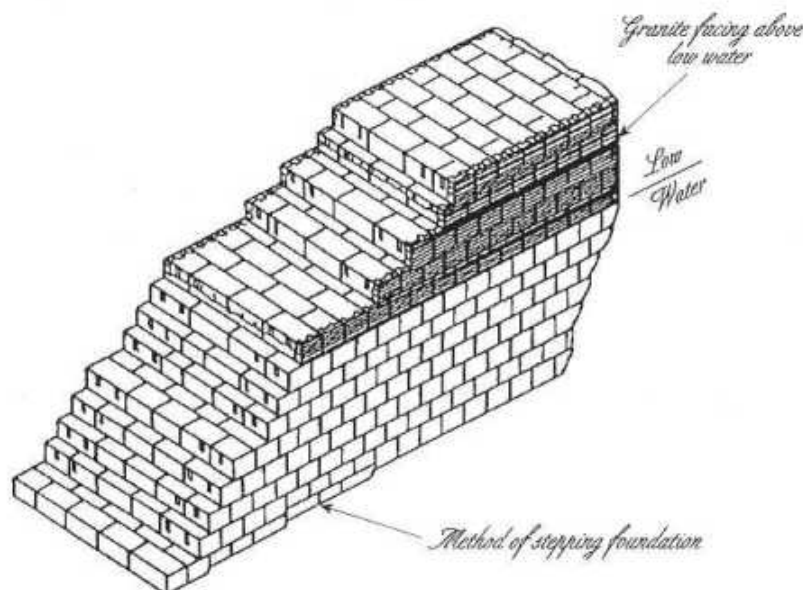
At Alderney, the lowest intended level of foundation for the wall along the outer sections of the breakwater was 7.3m (24ft) below low water on spring tides, but settlement or consolidation of the rubble mound caused this to increase to 9.1m (30ft) towards the seaward end. Towards the landward end, the foundation was set no more than 3.5m below low water. Large blocks of stone (later of concrete) were laid on the rubble after it had been allowed to settle for about 6 months. The batter of the wall of 2V:1H at the inner end is shallower than for many other contemporary breakwaters, and was revised to steeper slopes for the outer sections at Alderney. The wall at St Catherine's was battered at 3:1, and at Aberdeen at 8:1. The discussion by Marth et al (2004) on the effect of batter in providing restraining forces on the lower blocks is again instructive.

Blocks facing most of the breakwaters considered here were generally of dressed stone. Typical sizes are in the ranges 1m x 0.3m x 0.5m up to 2.5m x 1m x 1.5m. The sizes used were strongly dependent on the stone available in quarries near to the site,

and the stone-working skills available. Fine tolerances were possible, but would generally have been reserved for elements on the top of the breakwater, where easily seen. Facing stone on the breakwater wall would be dressed to give joint gaps typically less than 25-50mm. At lower levels, where inspection was more difficult, and placing times shorter, tolerances may have been wider, and joint gaps of up to 75mm might be expected.

Once production of concrete blocks became economic, block sizes increased dramatically, sometimes approaching 400 tons. Stoney (1898) records the use of blocks of approximately 3.5m x 6m x 7m for quay construction in 1871, and suggests their use at Alderney. It was clear however that the costs of equipment to produce, move and place such blocks (see Figure 3 and 4), necessarily restricted their use to large projects.

Gaps between adjoining blocks would generally have been negligible where blocks were laid in mortar. Mortar will however have deteriorated over the structure life; the joints then open up, allowing water into the hearting or core, and sometimes allowing blocks to move. Many failures of such walls have been associated with the loss of bond / filling between blocks. The use of concrete blocks, e.g. at Dover, see Figures 3 and 4, avoided many of the problems of bonding stonework, and made it much easier to make special provisions for joining blocks.



**Figure 3 Blockwork construction at Dover, 1898-1909,**

#### 4. Caissons (an aside)

Caissons have relatively rarely been used in the UK, although they are now often used by British designers working overseas. An early use of caissons by British engineers described by Routh (1912) was the construction between 1663 and 1684 of the main breakwater or Greate Mole at Tangier. Protection was needed for naval vessels supplying the garrison. The Mole was started in conventional fashion, with rubble foundations placed ahead of blockwork wall construction, starting in August 1663. The Mole had reached only about 350m over the next 5 years to August 1668, with delays caused by adverse wave conditions; loss of fill into the (soft) sand bed; the workforce who were often diverted to military duties; difficulties in obtaining materials; and very significant delays in payment for the work completed.

After the contract was re-negotiated, the contractor returned to site in April 1670 to find that the blockwork walls had been damaged and breached in at least two places by wave attack. The construction method was re-considered, and a type of caisson construction was proposed using "great wooden chests" bound in iron, and filled with stones and mortar or concrete.

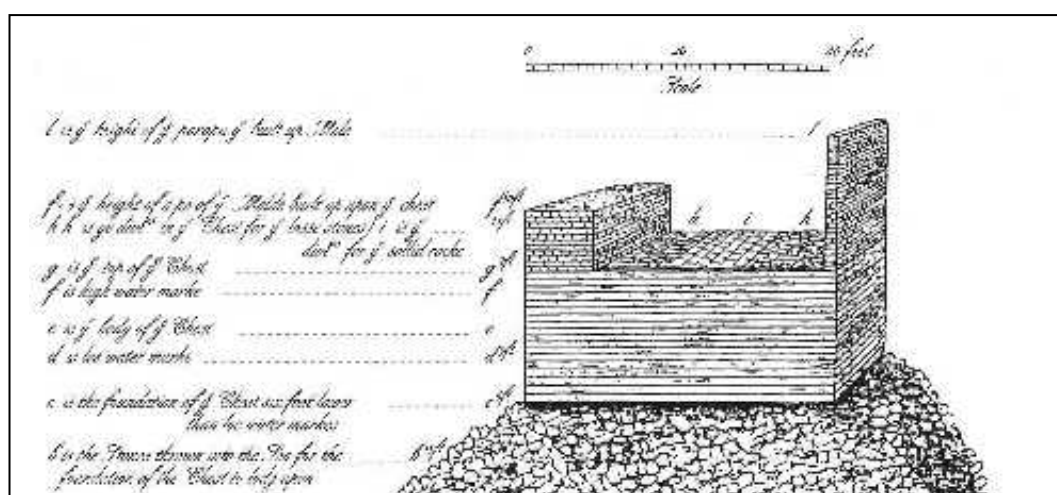
After much debate, and consideration of the new breakwater at Genoa, a new contractor was appointed to extend the existing structure using caissons.

Wooden caissons of 500 to 2000 tons (so large that they were named as ships) were towed out from England. On site they were sunk onto the rubble foundation being filled with stone bonded with a local mortar, Figure 5. Progress was now less subject to damage, and continued until 1678, when Tangier was attacked and all energies were diverted to its defence. Peace was concluded in 1680. It was then suggested that the British occupation be lifted, and therefore at the breakwater should be destroyed, lest it provide shelter to a later enemy. This demolition was completed in 1684 with much more difficulty than anticipated, and marked a halt in the use of caissons by British engineers until the late 1700's or later.

The largest use of breakwater caissons in UK was the construction of Brighton marina using about 110 cylindrical caissons, see Terrett et al (1979) and Scatchard et al (2009), although caissons are now used at locations around the world, see also Young (2009).



**Figure 4** Blockwork construction at Dover, 1898-1909,  
(photo from Dover Harbour Board)



**Figure 5** The “great chest” used at Tangier, 1677 (after Routh)

## 5. Failure modes for blockwork walls

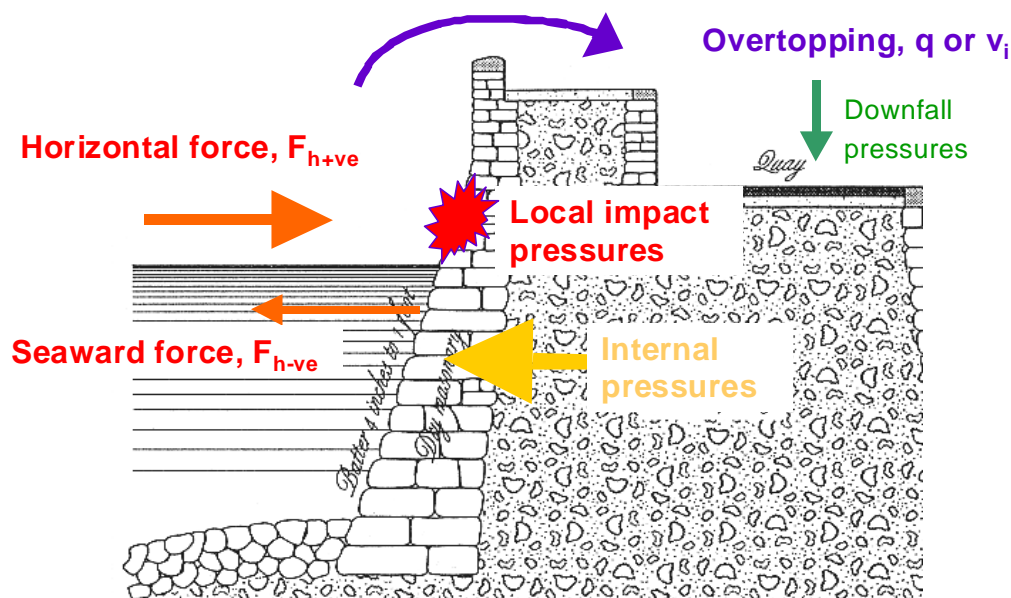
Very few design / analysis methods have been available for these breakwaters, indeed their original designs would have been based on vernacular rules derived from previous failures, see e.g. Paxton (2009). Some guidance would have been extracted from such failures, but few data, and fewer design rules are available in the forms used in modern design work. The failure modes which the structures were required to resist may be summarised:

- 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 continuity of the blockwork.

Of these, sliding or overturning of single elements (a), and gross foundation failure (b), have been relatively rare in the UK, although not unknown. Local failures leading to loss of continuity, and thence to overall failure, have been more common.





**Figure 6 Summary of wave-driven loadings on blockwork breakwaters**

The main forces acting on these walls arise from wave momentum and impact pressures, which in turn induce up-lift and internal water pressures. Overtopping may induce down-fall pressures. Geotechnical forces from backing materials; seismic; vessel and equipment loads; may also act. Breakwater walls resist wave action and geotechnical forces by their own weight, and by friction with underlying materials. Interlock or bonding between elements is essential to retain continuity and avoid loss of blocks and/or fill, as the constitutive elements are easily moved by wave forces if not acting together. A useful discussion on load transfer within a simple blockwork wall is given by Marth et al (2004) who demonstrate that the “spreading” angle for loads can be remarkably narrow, thus favouring very steep batters.

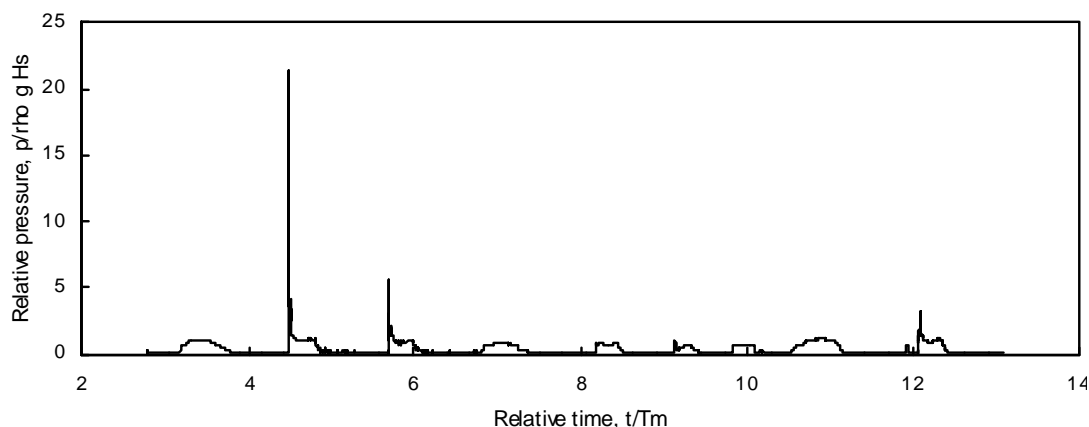
### Historical basis

The analysis methods for breakwaters before 1900 were based on experience derived from trial and error. Design formulae were not known, and would have been of little use as the design wave conditions were seldom well estimated. Publications between 1850 and 1900 give information on what was constructed, and what happened to it, but rather less on local

wave conditions. They describe examples of breakwater failure, but often fail to give a clear picture of cause and effect, as may be seen from discussions on the failure at Wick, see Paxton (2009).

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 after 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. This approach however suffered two major flaws.

Firstly, it ignored the influence that the wave reflected back from the wall itself might have 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 (e.g. Alderney) the wall was high, wave attack severe, and erosion of the mound could lead quickly to loss of support to the blockwork.



**Figure 7 Example impulsive wave pressures (dimensionless) on vertical wall with small toe mound, from Allsop et al (1996), McKenna (1997)**

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 5-10 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 Marth et al (2004).

### Stability / movement of elements

Direct impact pressures will not cause distress to good quality stone blocks per se, but 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. Mortar joints will gradually deteriorate as they lose strength and fine materials are washed out. This will accelerate where wave impacts are sufficiently heavy to cause movement of the wall, or where the foundation mound moves. In time, wave pressures will penetrate around the block, and will reach

the hearting material behind the face. Experience has shown that voids then develop within the wall, often unseen by any inspecting engineer.

Little information is available on the stability of individual blocks against wave forces. The main wave pressure acts on the outer face, tending to push the block into the wall. Example measurements of wave pressures (quasi-static and impulsive) on a vertical wall are shown in Figure 6, after Allsop et al (1996), McKenna (1997). The longer duration loads shown here are well-predicted by formulae by Goda (1985, 2000). The short duration impact pressures are highly variable, although research within PROVERBS (see Oumeraci et al, 2001), or by Cuomo et al (2004, 2005) have suggested prediction formulae for impulsive wave pressures.

In the previous analysis by Allsop & Bray (1994), a method by Partenscky (1988) based on large wave flume tests suggested that impact pressures of very short durations (0.01 to 0.03s) might be calculated from:

$$p_{\text{dyn}} = K_L \rho g H_b \quad (1a)$$

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

$$K_L = 5.4 \left( \frac{1}{a_e} - 1 \right) \quad (1b)$$

Blackmore & Hewson (1984) conducted field measurements at sea walls in the UK, from which they developed a model based on momentum exchange. Impact pressures  $p_i$  depend on the shallow water wave velocity,  $v_c$ ; the wave period,  $T$ ; and an aeration factor,  $\square$ , which depends on the roughness of the foreshore:

$$p_i = \lambda \rho T v_c^2 \quad (2a)$$

Values of  $\square = 0.3$  for a rough and rocky seabed, and  $\square = 0.5$  for a regular seabed are recommended. The breaking wave velocity might be calculated from the breaking water depth,  $h_b$ , and breaking wave height,  $H_b$ :

$$v_c = [g(h_b + H_b)]^{0.5} \quad (2b)$$

Once impact pressures can propagate into the wall, the stability problem has changed, the block is bounded by the surrounding blocks, the open sea, and an internal void. Pressures communicate through gaps between blocks, and will act on the back face of the block. For blocks that are not held by those surrounding, a sliding restraint might be given by the (buoyant) self-weight times the friction coefficient, say  $\square=0.7$  (see Cornick, 1969). The propagation of wave pressures into cracks is discussed by Muller (1997), Wolters et al (2004), and by Marth et al (2004).

Improved methods to predict the occurrence, magnitude and durations of impulsive loads have been discussed by Allsop et al (1996), Cuomo & Allsop (2004) and Cuomo et al (2009).

### **Idealised analysis example**

Allsop and Bray (1994) considered an idealised block, 2m deep x 1m across x 1m high, attacked by breaking waves of  $H_b=5m$ . The block was restrained only by its self-weight, and frictional resistance against the block below. Their calculation used impact pressures estimated by equation (1). Impact pressures would then be transmitted through the joints to the void behind the block. In water with air content of 10 - 30%, the speed of sound in the air/

water drops to approximately 30m/s, slowing the 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 out across the block may reach a significant proportion of the original impact pressure. They speculated that, even if severely damped, a net outward pressure of more than 0.2 of the original impact pressure would not be unreasonable.

The block will be restrained by its self-weight of 5.3 tonnes, 3.2 tonnes in water, giving a frictional resistance of about 2.2 tonne. The force to overcome this would be generated by a net pressure difference across the block of no more than 20 kN/m<sup>2</sup>. For well-aerated wave impacts, the pressure applied to the front face could well exceed 600 kN/m<sup>2</sup>, suggesting that even relatively mild wave impacts could move such a loose block. Ten years later, Marth et al (2004) report experiments illustrating the same process and confirming these conclusions.

This simplified analysis identifies a mechanism by which a block can be extracted by wave impacts under relatively mild conditions, once the integrity of the seaward facing allows penetration of impact pressures through the facing. These effects are further compounded if fine materials can be washed out from the hearting leaving significant voids within the structure.

### **Wall / foundation interaction**

The extraction of blocks from a vertical breakwater wall can also be occasioned, or accelerated, by any local loss of support for the wall foundation. Even quite small movements of the base of the wall may be sufficient to precipitate cracking of pointing/ mortar between blocks. This may be due to local settlement of the mound; or loss of lateral restraint allowing the seaward and landward toes of the wall to spread.

Little guidance is available to analyse these (potential) failure modes. The major difficulties arise due the small foundation movement that may precipitate wall failure, and the simplicity of the methods available to analyze the stability of rubble mound foundations under wave action. The simplest approach is to assess the condition for which mound material can be moved by wave action using prediction graphs e.g. those by Brebner & Donnelly from hydraulic model tests reproduced in the Shore Protection Manual (1983), and can be used to estimate the lowest size of rubble for particular wave/ water level combinations. More recent methods for rubble mound breakwaters based on data from Japan are presented in the CIRIA/ CUR/ CETMEF rock manual (2007), but these generally envisage much more movement than could be allowed for the foundation of blockwork walls.

Local consolidation or settlement of the rubble mound is also extremely difficult to predict to the levels of precision that would be required to analyze the risk of precipitating any (local) movement of a blockwork wall.

### Design formulae

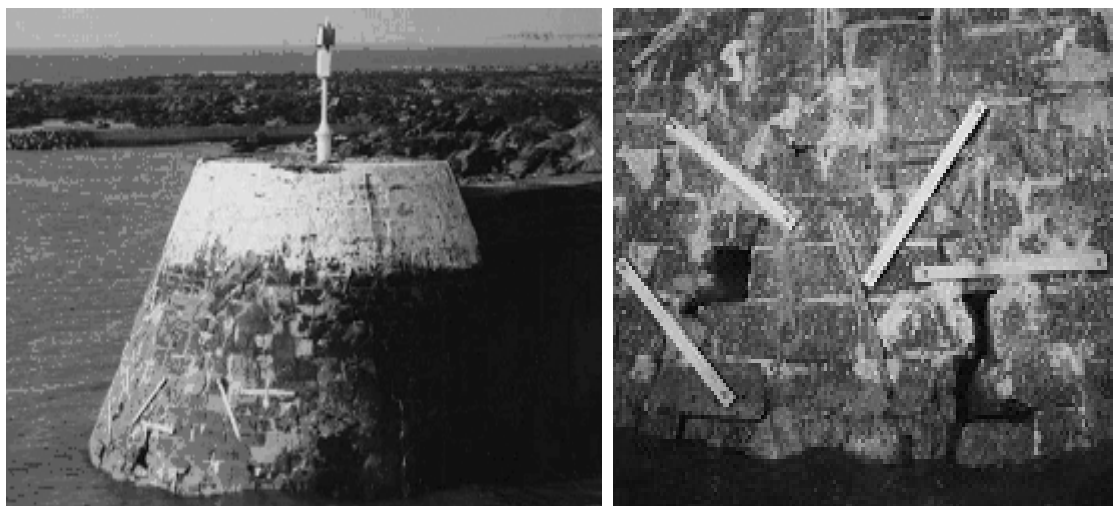
Few design formulae are available for the structures discussed here as design methods prior to circa 1900 concentrated on identifying configurations and procedures that had previously been found effective. Sainflou (1928) developed a formula for wave pressures, but not for the breaking conditions that cause most of damage to these structures. The development of further understanding of breaking wave pressures, e.g. by von Karman, Bagnold, and others are discussed by Blackmore & Hewson (1984), and Müller (1995, 1997).

The main formulae used in the design of vertical breakwaters are those developed by Goda (1985, 1995, 2000) to calculate effective sliding loads on caisson breakwaters. Some designers have also used the method by Minikin (1963) to estimate wave impact pressures. The Goda

formulae were developed for vertical caisson breakwaters, typically of 1,000 to 10,000 ton, and therefore un-responsive to very brief impacts, even if reaching large peak pressures. The method calculates quasi-static equivalent pressures, and was calibrated against laboratory and field experience of caisson displacements. Since its derivation, the method has been further validated by van der Meer et al (1992) who showed good agreement with measurements from site specific model tests conducted at Delft Hydraulics and Danish Hydraulic Institute.

The Minikin formula extended the theoretical model by Bagnold, calibrated by measurements of wave impact pressures at Dieppe by Rouville et al (1938). This method is cited by the Shore Protection Manual, but is not well-supported by further site or laboratory data. Readers should also be cautioned that versions of Minikin's formulae have been corrupted ( $\pi g$  taken as 101, which can only be true Imperial units where  $g = 32.2 \text{ ft/s}^2$ ). Partenscky (1988) proposed pressure distributions based on momentum exchange and tests at large scale in the Large Wave Channel (GWK). Peak pressures of about  $300 \text{ kN/m}^2$  for a breaking wave of  $H_b=1.5 \text{ m}$  are contrasted with estimates of about  $100 \text{ kN/m}^2$  using Minikin's method.

Recent studies reported by Cuomo and co-workers (2004, 2005, 2009) have developed new prediction methods for the magnitude, and duration of impulsive loads based on small and large scale testing. There do however remain considerable uncertainties in prediction methods for the magnitudes and spatial distribution of impact pressures, and of their probability distributions. These uncertainties have often been compounded by uncertainties in the dynamic response characteristics of the structures / elements concerned. New methods have been reported for large caissons, but no attention has yet been paid to the responses of smaller elements, particularly those forming the older types of vertical walls.



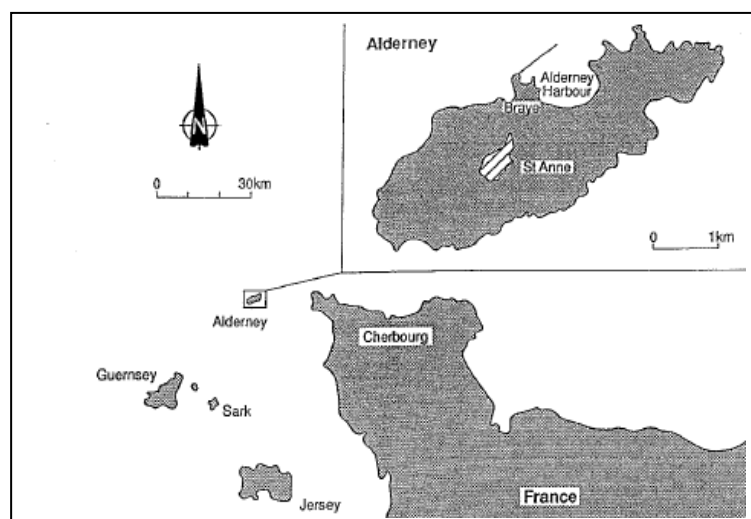
**Figure 8 Outer end of Arbroath West Breakwater, and detail of damaged / missing blockwork**

## **6. Deterioration and/or failure**

### **6.1. Data on performance**

Relatively little information on the performance of breakwaters was derived by Bray & Tatham (1992). Of those owners from whom information on breakwaters was requested, only 8% responded, perhaps suggesting that these structures have given relatively little cause for concern in recent years. In the CIRIA report however, it was noted that incremental degradation of such walls was often overlooked, and that the apparent lack of problems might be primarily due to lack of inspection. In some instances, it might be concluded that any damage was either so rapid that the structure was abandoned, repaired, or it was replaced at a relatively early stage in its life. In other instances, it might be concluded that historical rates of deterioration have been so slow that the return period for any significant expenditure is quite long. This would however ignore the brittleness of the failure modes for many of these structures. The CIRIA project concluded that there is a significant requirement for inspection and monitoring to avoid those sudden failures that occur when the structure has degraded to a failure point. This is further illustrated for St Catherine's breakwater by Hold (2009).

Various papers reviewed by Allsop & Bray (1994) give details of breakwater performance, but often fail to distinguish clearly between cause and response. A good example of this problem is given by reports of damage to Wick breakwater. Stevenson (1874) describes the start of breakwater construction in 1863 using dry-placed blocks of 5 to 10 tons. During storms in 1870, a section of about 380 ft (115m) of the breakwater was destroyed, presumably by breaching the breakwater wall. This section was then rebuilt using Portland cement to bond the block facing, and iron dowels between courses. A storm in February 1872 gave wave impact pressures so severe that facing stones were shattered, although Stevenson's report does not identify whether this was by direct wave impact, or could have been by stones from the mound being hurled against the face (see discussion on Alderney). In December 1872 a section of blockwork bonded together and estimated as weighing 1350 tons was pushed into the harbour. This was followed for another section weighing 2600 tons in 1873, cited by Cornick (1969) as evidence of impulsive forces. Shield (1895) however suggests that damage was strongly influenced by foundation failure, but gives little other information. More details on the construction and failure are given by Paxton (2009).



**Figure 9 Alderney and the Admiralty breakwater**

## 6.2. Alderney breakwater

### Background

Alderney is a small island off the coast of France exposed to Atlantic storms (Figure 9). The tidal range is 5.2m, and tidal currents may exceed 7-8 knots in the Race of Alderney. Waves from the Atlantic reach the island with relatively little reduction. The Admiralty Breakwater was built to shelter Braye Harbour for the British Navy. Construction of the breakwater, designed by James Walker (who also designed St Catherine's breakwater), started in 1847. The design included a mound to low water, surmounted by blockwork walls with rubble infill (Figure 2), all to be quarried on Alderney. The breakwater wall, often termed the sea wall in historic accounts (see Vernon-Harcourt, 1873), was laid without mortar, and without cement in the hearting.

During construction, it became clear that the design was insufficient to resist wave actions so the section geometry was revised. The foundation level was reduced to 3.5-4m below low water. Foundation stones were laid in cement mortar. The wall batter was steepened, and the filling was concreted. This construction continued to 823m by 1856. The section was then further revised, again reducing the level of foundation stones, and steepening the wall

face. Construction of the outer section was completed in 1864, giving a total length of 1430m.

Between 1864 and 1870 the breakwater was damaged each winter, including a number of breaches through the wall. By 1870 the Admiralty need for the harbour had reduced, and further works at Alderney were not merited. Thereafter the wall was protected by stone dumped to maintain the foreshore, and reduce the potential for movement of the wall foundations. About 300,000 tons were tipped in front of the wall between 1864 and 1871. But even this proved too costly and from 1873, repair and maintenance work covered the inner length of 871m only. The outer portion was abandoned, and its wall collapsed leaving a mound about 4m below low water. For the shortened section, approximately 20,000 tons of stone were dumped annually, and further work was still required to repair breaches in the superstructure. Dumping of rock ceased in 1964. Responsibility for Alderney breakwater was transferred to the States of Guernsey in 1987.

At the breakwater, Atlantic storm waves reduce relatively little with the 1:50 year offshore wave condition of  $H_s=11.0\text{m}$  reducing to  $H_s=8.0$  to  $8.5\text{m}$  at the breakwater. Wave impacts onto the breakwater wall are increased by the mound

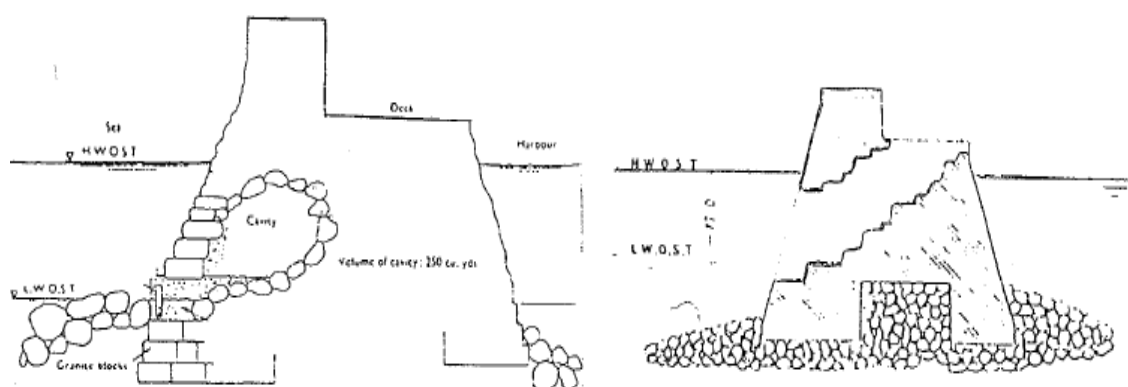
as waves break directly against the wall. Storms usually persist for many hours, so the breakwater is exposed to most combinations of wave height and water level, including those which give impulsive breaking against the wall. Direct impacts shake the breakwater, and crack the pointing and mortar joints. The pressures force water into the blockwork joints and any voids behind. Loose rock from the mound is thrown against the wall as missiles, and has abraded the wall by more than 1m. The typical rock in the mound has reduced, and the mound has filled with sand sized material from the abrasion. The total volume has reduced since end of dumping in 1964.

The breakwater wall has continued to suffer repeated damage, and maintenance costs for years up to 1991 were estimated at around £500,000 per annum, excluding costs of storm damage. A team of 8 men re-pointed the wall above mid-tide level each summer, filled cracks and replaced damaged masonry each summer. A team of 6 engineering divers worked on repairs to the toe of the wall, both above and below low water.

During 1989/90, storms battered the breakwater for six weeks. At its peak on 25/26 January 1990, the storm had a return

period of about 1:25 years, with offshore conditions of  $H_s=10$  to 10.5m. During the next six days the storm subsided slowly, then rose again to  $H_s > 7$ m. On 11 and 12 February 1990, waves again exceeded  $H_s = 9$ m. This continuous pounding cracked the masonry facing, and a large cavity was formed in the wall. This was breached by an explosive failure clearly audible around the island. Examples of similar breaches in Figure 10 are shown by Vernon-Harcourt (1885) and Bishop (1950). An emergency procedure had previously been formulated, and repair work costing £1.1 million was started within 10 days.

The main failure mode of the wall is by removal of individual blocks, followed by erosion of finer material from behind the blockwork. The integrity of the blockwork depends upon the exclusion of wave pressures, and on frictional resistance between blocks. Where the foundation support offered by the mound is locally insufficient, or the mortar joint between blocks deteriorates, blocks are relatively easily extracted. Experience shows that local failures spread very rapidly, and are often followed by failure of the deck slabs and/or harbour wall. The damage caused in January and February 1990 was therefore quite typical.



**Figure 10 Example breaches to Alderney breakwater upper wall**

## 1990 investigations

Coode Blizard and HR Wallingford, appointed by States of Guernsey, formulated a staged long term strategy. Stage 1 reviewed survey and historical data, and inspected the breakwater. Waves were modelled and currents measured in Stage 2. Detailed work in Stage 3 analysed materials on the breakwater mound and adjoining beaches. Hydraulic and geotechnical studies examined the behaviour of the breakwater under a wide range of conditions to explore the most critical damage mechanism, and to identify measures to reduce damage.

Stage 1 showed the breakwater walls to be in good condition, and wall surveys 1984-1989 showed movements of the wall generally less than 10mm. Local spreading of the sea and harbour walls by about 50mm coincided with a rock outcrop beneath the mound.

Surveys of the mound re-drawn at constant scale and datum showed however that substantial volume had been lost from the seaward part of the mound since 1970. The upper foreshore had fallen, but more surprisingly considerable volumes had been lost from the lower slopes. Diving surveys confirmed that the mound material ranges from boulders to sand, and the lower slopes are well filled with sand. Excavations on the upper mound at the wall showed that voids in the rubble were also well-filled by sand originating mineralogically from abrasion of the wall and the mound. Analysis of mound surveys from both upper and lower slopes between June 1970 and August 1990 indicated loss of material averaging 3200 m<sup>3</sup> per year over the breakwater. HRW re-surveyed the mound in June 1993 with an average loss 1990 - 1993 of 5800 m<sup>3</sup> per year. Over 1970 - 1981 the loss had averaged only 2350 m<sup>3</sup> per year; then the loss over 1981 - 1993 increased to 4450 m<sup>3</sup> per year.

Wave conditions at the breakwater were derived from a refraction model which included currents. The largest waves at the breakwater are at mid-ebb, with the 1:50

year return giving  $H_s=8.0$  to 8.4m and  $T_m = 12.7s$  from 320 - 330°N, nearly normal to the structure.

## Geotechnical studies

Current close to the lower mound are insufficient to erode the volumes / sizes concerned. Two other failure modes were: a slip of the outer face, occasioned by increased pore pressure; or the loss of fine material from the mound, prompting local collapses of the foundation to the wall. Each failure mode was explored using empirical design methods, laboratory experiments and analytical calculations.

When first built, the mound had side slopes with a safety factor of 1.0 set by the geotechnical strength of the material. The original material was well-graded. The strength parameter  $\phi'$  would be near 40° for low stress states, and 1870-2 profiles indicate slopes of the lower face of 36-38°. Surveys in 1905 indicate that a significant change had occurred with the seaward slope flattened to about 28°. Over the intervening period, additional material had been dumped, and much of the mound had become filled with abrasion products, reducing its void porosity and permeability. The mixture of sand and stone might therefore have been expected to have adopted a slope at about 28°, rather shallower than the original 36-38°. Flow through the outer layers of the mound would now be impeded, and pore pressures would increase under wave attack. Even if the drained strength of the filled rubble could be as high as  $\phi' = 35^\circ$ , undrained failure would take place at values much lower than this. A reasonable value for  $\phi'$  mobilised for undrained failure would be 27° to 29° for  $\phi'_{max} = 35^\circ$ .

## Conclusions of the 1990-91 Alderney investigations

The response of Alderney breakwater was studied using field and laboratory measurements, numerical and physical models. Rates of erosion from the mound were quantified and showed erosion rates



that have increased in time. Loss of material from the lower slopes of the mound has been ascribed to local geotechnical failures of the sand-filled mound. These are expected to continue, as will local lowering of the upper slope of the mound. These studies confirmed that it is essential to stabilise the mound (both lower and upper slopes), to ensure stability of any protection on the upper slope.

## 7. Guidance for analysis

Few or no codified design rules were available for these breakwaters. Some empirical information might be extracted from analysis of performance, but very little information, and fewer design rules were available in the forms used in modern design work. Most designs were site specific, supported by rules derived from experience. The main failure modes may be summarised:

- a) Sliding or overturning of the breakwater wall as a single entity – analysed by sliding / overturning loads vs. self-weight;
- b) Gross (slip) failure of the rubble mound foundation, allowing whole body movement of the wall – analysed by slip circle methods supported by measurements / calculations of mound pore pressures;
- c) Removal of blocks from the wall, resulting in a loss of continuity, and hence destruction of the wall – depends on impulsive pressures, continuity of blockwork and internal voids / permeabilities;
- d) Local failure of the mound (scour)

allowing movement of blocks, loss of fill and/or continuity of the blockwork.

## Acknowledgements

This paper develops an earlier version compiled for a workshop at Port and Harbour Research Institute in Japan which drew on results of research supported by MAST I and II programmes of the European Union, and at HR Wallingford by the (then) Construction Policy Directorate of the UK Department of Environment. Presentation and publication of that paper was supported by HR Wallingford and the University of Sheffield in the UK, and the Port & Harbour Research Institute and JISTEC in Japan.

Historical information on vertical walls cited in this paper was provided as part of the research project on "Old Waterfront Walls" completed by Bray & Tatham (1992) for CIRIA. Permission to use this information is gratefully acknowledged.

The author is grateful for advice and assistance provided by Mike Chrimes, librarian of the Institution of Civil Engineers, in researching some of the historical information cited in this paper; and to Gerald Müller and Janice McKenna (then of Queen's University of Belfast), Adam Bezuijen (then of Delft Geotechnics) and Tom Bruce (Edinburgh University) for guidance on impact loadings, internal pressures, and historical structures. The author would also like to express gratitude to the BloCSnet and VIFSNet research networks.

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