# CHAPTER 194

## New design methods for wave impact loadings on vertical breakwaters and seawalls

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**ABSTRACT**

This paper discusses wave impacts on vertical and composite breakwaters and related coastal structures. It describes types of vertical walls in use, with particular reference to older walls that may be much more influenced by wave impacts. Methods to estimate wave forces are identified. Analysis of performance suggests that these under-predict wave impact loads, and cannot identify combinations of geometry and wave conditions which lead to impacts.

Comprehensive 2-dimensional hydraulic model tests have been conducted using random waves to measure wave pressures (and other responses) on a wide range of simple and composite vertical walls. The test results have been used here to:

- Identify the ranges of geometry and wave conditions which lead to wave impacts;
- Develop a simple method to estimate wave forces under impact conditions.

Analysis of % of impacts has defined a new design diagram to identify wave conditions and wall / mound geometries which cause impacts. These results are intended for engineers analysing vertical or composite walls in deep water, in harbours, or along the shoreline.

## 1. VERTICAL WALLS

Seawalls or breakwaters around the world have often been built with vertical or steep faces formed by small blocks joined together. The structure relies on its weight to resist sliding or overturning forces, and on the bonding or jointing of the blocks to maintain its monolithicity. The integrity of blockwork walls depends on their resistance to local pressures or pressure gradients. Modern structures may be formed from larger elements, perhaps full-depth cellular caissons filled with sand or rubble, and founded on rubble. A few modern structures use concrete blocks bonded or keyed together, or thin concrete elements.

Much of the historical and experimental information discussed in this paper has been presented in the comprehensive research report by Allsop et al (1996a).

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**Construction of breakwaters, piers, and seawalls**

Unless founded directly onto rock, vertical breakwaters or piers built before about 1900 used rubble slightly below low water, and surmounted by blockwork walls. Hewn stones were laid in bond, generally slightly off vertical. Blocks were originally laid dry, or in lime / pozzolanic mortar. Cement mortars were used after about 1900, and concrete blocks after about 1880. Tensile, bending, or shear loads were transferred between adjoining blocks, or courses of blocks, by iron cramps, keys or joggle joints between blocks.

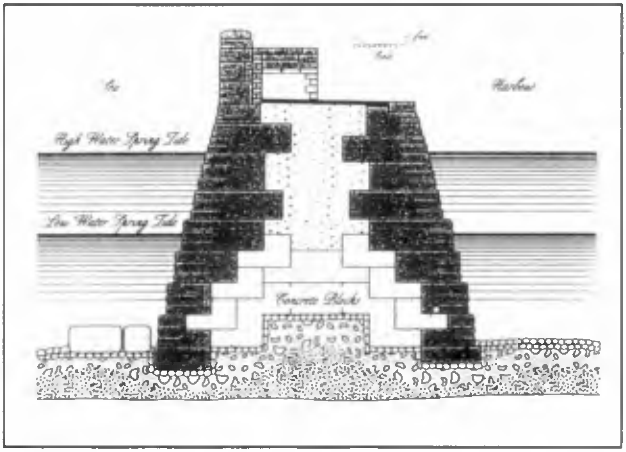

**Figure 1 North Tyne Breakwater**

Concrete blocks were used at North Tyne in 1855 (Fig 1), for Dover breakwater, 1866, and at Cork in 1877. Concrete bags formed a foundation at Fraserburgh in 1877, and for Ardrossan Pier in 1892. Concrete filling was used for the later stages of Alderney breakwater 1849-1866, Aberdeen south breakwater, 1873; North Pier at Aberdeen, and the Fraserburgh breakwater, both in 1877. The Italian engineer Coen Cagli re-introduced vertical wall breakwaters to Italy after a visit to Britain in 1896 where he saw blockwork breakwaters at Dover, Sunderland, North Tyne, Peterhead, and Wick.

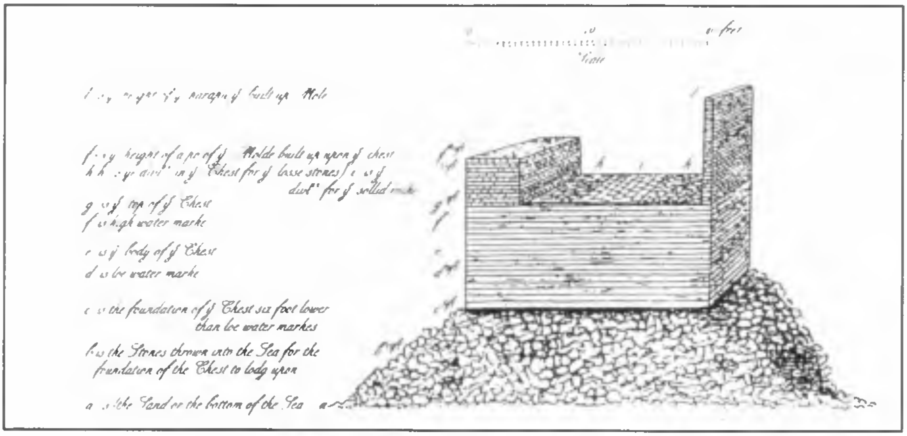

**Figure 2 Timber caisson used for the Greate Mole at Tangier, 1677**

Caissons were rarely used in the UK before 1900, but construction of the Greate Mole at Tangier by British engineers using caissons in 1660s is described by Routh cited by Allsop et al (1996a). The Mole was started with rubble foundations placed ahead of blockwork in August 1663, but only reached 350m by August 1668 due to adverse wave conditions; loss of fill into the sand; diversion of the workforce to other (military) duties; difficulties in obtaining materials; and significant delays in payment for work completed. Construction re-started in April 1670 with the blockwork walls damaged and breached by storms. A new construction was copied from Genoa using "great wooden chests" bound in iron, and filled with stones and mortar. After much debate, some reported in Samuel Pepys' diaries, wooden caissons of 500 to 2000 tons were towed out from England, and sunk onto the foundation filled with stone bound in mortar (Fig 2). Progress was quicker with the new construction and suffered less delay than the blockwork sections. In 1680 however, peace was concluded, and the breakwater was then destroyed lest it provide shelter to a later enemy.

The development of many harbours around the UK between 1850 and 1900, and their survival, have reduced the need for new harbours around the UK, so relatively few have been constructed since 1900. Vertical breakwater exceptions to this are at the new marina harbour at Brighton protected by circular caissons based on those used at Hanstholm in Denmark; and the vertical wave screen breakwaters at Sutton Harbour, Plymouth, and Cardiff Barrage.

**Performance in service**

The life of a breakwater may be considered in three periods: construction; initial service; and extended service (often well beyond that used in design life calculations). Most damage occurs early in its life, even during construction, so most breakwaters which survive the first 5 years without damage are likely to survive the next 40-50 years. This simplification however ignores steady or accelerated deterioration which may lead to sudden failures in later life.

Stevenson describes construction at Wick started in 1863 using dry-placed blocks of 5 to 10 tons. During storms in 1870, about 115m was destroyed, presumably by breaching the wall. This section was then rebuilt using cement to bond the block facing, and iron dowels between courses. A storm in February 1872 gave impact pressures so severe that facing stones were shattered, although Stevenson does not identify whether this was by direct wave impact, or by stones from the mound being hurled against the face. In December 1872 a section of blockwork bonded together and estimated as weighing 1350 tons slid into the harbour. This was followed in 1873 by movement of another section of 2600 tons.

During construction of Catania breakwater in Sicily in 1930, large blocks slid backwards into the harbour under wave attack. The damage was repeated in 1933 when much of the upper part of the breakwater slid backwards, due to a lack of horizontal connectivity between layers. Later structures in Italy included connections to resist horizontal forces, but few if any existing structures were re-appraised or strengthened, and collapses continued at Genoa in 1955, at Ventotene in 1966, Palermo in 1973, Bari in 1974, and Naples in 1987.

Mutsu-Ogawara port on the Pacific coast of Japan was under construction in 1991, when it was hit by waves of $H_s$=9.9m, substantially above the 1:50 year design condition of $H_{sd}$=7.6m. Damage was particularly severe where mounds of armour intended to cover the front face were incomplete and/or had already been damaged. Waves tripped over the part-height mounds causing impact forces so severe that two 24m long caissons suffered significant structural damage, one of them losing most of its upper part.

Sakata port is on the Japan Sea, less exposed than the Pacific coast. Even so, waves during winter 1973 / 74 reached $H_{so}$=7.2m and exceeded $H_{so}$=4.5m on 4 other occasions. In depths no more than 9-10m, these conditions would have reached or exceeded the breaking limit. Nearly all 39 caissons, each 20m long and 17m deep, slid during these storms, some by 4m.

In December 1990, a small breakwater was damaged at Amlwch, North Wales. It is about 60m long, slightly curved in plan, and is constructed using concrete blocks in slices on mass concrete on the rockhead. The outer end of the breakwater was slid backwards by about 0.3, cracking the crown wall in three places. Wave conditions are estimated as $H_{so}\geq$4m, with

$T_m$=9s. The foreshore is very steep, approximately 1:13, falling outside of any established design method. The depth at the toe probably reached at least 11-14m. Allsop & Vicinanza (1996) estimated inshore wave conditions limited by depth to $H_{si}$=4m at MHWS, but reducing to $H_{si}$=3.6m at MLWS. Using the method of Vicinanza et al (1995), $F_h$ was calculated as 1040kN/m at MHWS. With no up-lift force for blocks direct on concrete, and $\mu$=0.5, these give a factor of safety of $F_s$ = 0.9 at high water, contrasted by predictions using Goda which gave $F_s$ = 1.2 at high water, and $F_s$ = 2.3 at low water.

In late October 1966, the blockwork breakwater of Porthcawl in South Wales lost its capping wall over a length of about 55m, again probably due to a single direct wave impact.

These examples demonstrate that wave impact forces are very high, and the more recent damage suggest that predictions of wave loadings / responses remain very uncertain. As many blockwork walls approach 150 years old, and the potential for local or catastrophic collapse increases, it becomes more important to re-examine wave load prediction methods.

## 2. WAVE LOADINGS ON VERTICAL AND COMPOSITE WALLS

The main loadings acting on these types of walls arise from: direct wave pressures; up-lift forces; quasi-hydrostatic forces from internal water pressures; and geotechnical forces / reactions from backing or supporting materials. These structures resist wave and geotechnical forces essentially by their own weight, and by friction with the underlying materials. Interlock or bonding forces between component elements maintain continuity and avoid movement or loss of elements and/or fill.

The simplest failure mode for monolithic vertical structures is sliding under direct wave forces, primarily under horizontal loads, but also influenced by up-lift forces. Failure by overturning may be examined by assuming rotation about the rear heel of the caisson / wall, but the point of rotation depends upon bearing capacity and geotechnical characteristics of the mound / foundation. Analysis of foundation failures has been discussed by de Groot et al (1995), and constitutes a major part of the MAST III research project PROVERBS. Blockwork breakwaters may also fail by loss of integrity where a block is removed (seaward) by net suction forces, followed perhaps by progressive damage and then catastrophic collapse.

It is often convenient to treat wave pressures / forces on these structures as quasi-static / pulsating; of dynamic / impulsive or impact, see also Figure 4 in Section 3.

Quasi-static or pulsating wave pressures change relatively slowly. A wave impinges directly against the structure applying a (quasi-) static pressure difference. The obstruction of wave momentum causes the water surface to rise up, increasing the force on the wall. The net force is related closely to the peak water level, and can be estimated using simple methods. Most design methods assume static loadings, and simple equilibrium conditions.

Dynamic or impact pressures arise where the wave breaks directly onto the structure. Impact pressures are substantially greater than pulsating pressures, and of much shorter duration. The processes of wave breaking are not well understood and the occurrence of breaking cannot be predicted with reliability, so these pressures are extremely difficult to calculate, and have historically not been used in design calculations. Schmidt et al (1992) remind us that there are still two attitudes to impact loadings. The first simply assumes that impact pressures are not important and need not be adopted in design. The second is to skip the problem of evaluating impact loads by assuming that the structure can be designed so that impacts will not occur. A third (newer) approach is to conduct dynamic analysis of the structure, its foundation, and the loads. This requires high levels of data on wave loadings, and on the geotechnical response characteristics of structure / mound / foundation, but is likely to become more frequently used.

These problems are compounded by uncertainties in defining conditions that lead to impacts. Schmidt et al (1992) and Oumeraci (1994) define 7 breaker classifications in terms of $H_b/d$, but the breaker height $H_b$ is extremely difficult to predict, so these classifications are of limited use. Klammer et al (1996) define 2 types of plunging and a "flip-through" breaker, but then derive a suggested force formula for all three types lumped together.

Goda (1985) described rules to identify whether particular structures or sea states will cause impulsive wave conditions, and that method is re-interpreted as Figure 3.

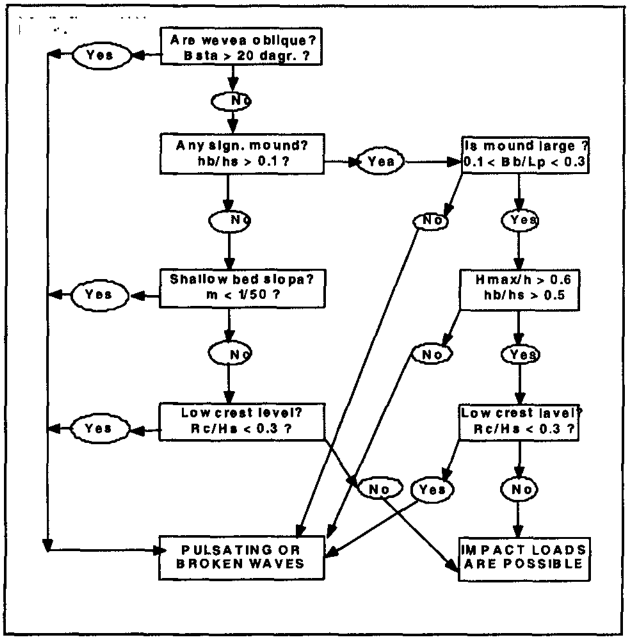


**Figure 3 Impact breaking conditions, derived from Goda (1985)**

**Wave force prediction methods**

Hiroi's simple formula gives a uniform wave pressure p on the front face up to 1.25H above still water level where H the wave height, and $p = 1.5\rho_w gH$. Sainflou's method calculates a maximum pressure, $p_1$ at static water level, tapering to zero at $H+\delta_0$ above SWL, and reducing linearly with depth from $p_1$ to $p_2$ at the rubble base. The Shore Protection Manual (1984) however suggests that Sainflou's method may over-estimate wave forces for short non-breaking waves, and uses Miche-Rundgren formulae to derive the clapotis height from which an (assumed) linear hydrostatic pressure is calculated. For long waves of low steepness, the SPM recommends Sainflou's method.

Goda's method

The most widely used prediction method for wave forces on vertical walls was developed by Goda (1985), primarily to calculate horizontal forces for concrete caissons on rubble mound foundations, and calibrated against laboratory tests and back-analysis of historic failures. This method assumes wave pressures on the front face are distributed trapaezoidally, reducing from $p_1$ at s.w.l. to $p_2$ at the caisson base. Above s.w.l. pressure reduces to zero at the (notional) run-up height $\eta^* = 1.5H_{max}$, where $H_{max}$ is taken as $1.8H_s$. The method describes impulsive and deflected wave components by coefficients $\alpha_1$, $\alpha_2$, and $\alpha_3$. The effect of relative depth to wave length on the pulsating component is represented by $\alpha_1$; the effect of impulsive breaking due to the mound is represented by $\alpha_2$; and $\alpha_3$ accounts for the relative crest level and relative depth over the toe. The depth h is taken at the mound toe, and d over the mound at the front face of the caisson, but $h_b$ is taken $5H_s$ seaward.

The caissons on rubble foundations considered by Goda had natural periods around 0.1 to 0.3s. When subjected to loads of much shorter durations, the effective load is smaller than the applied load. Thus for the short peak pressures caused by wave impacts, Goda's formula does not give actual pressures, but equivalent static loads for the dynamic system of caisson, mound and foundation. Goda noted that impulsive pressures caused by waves which break onto the wall may rise to $p=10\rho_w gH$, but judged that vertical breakwaters would not be designed to be exposed to direct impulsive pressures.

**Wave impact force predictions**

In Europe, engineers observed the effects of very large forces on some walls, and noted very short impacts coupled with very large pressures. Bagnold postulated a model of air compressed by the exchange of wave momentum, see Klammer et al (1996) for a simple description. At maximum pressure, all the wave momentum has been converted to pressure over the impact rise time. This approach however required identification of the thickness of the air pocket, and of the virtual length of the water piston, neither of which could be measured. Minikin's (1963) used Bagnold's model to develop to estimate impact pressures caused by waves breaking directly onto a wall, and therefore addressed the problems of impact pressures. The resulting expression for $p_{max}$ may be written:

$$p_{max} = \tfrac{1}{2}C_{mk}\,\pi\,\rho_w\,g\,H_{max}\,(1+d/h)\,(d/L) \qquad (1a)$$

where $C_{mk}$ is defined to fit Rouville's data, accounting for typical sizes of air pocket. Minikin suggests $C_{mk}$=2, which is then cancelled within eqn. 1a to give the simpler version used in the British Standard, BS6349 Pt1, BSI (1984):

$$p_{max} = \pi\rho_w g H_{max}(1+d/h)\,(d/L) \qquad (1b)$$

Unfortunately, this expression was re-written by Minikin with $\pi\rho_w g$ replaced by 2.9! This (mis)-use of dimensioned coefficients was later compounded by the Shore Protection Manual, and others. The total horizontal force may be written in dimensionally correct terms:

$$F_{hmax} = \tfrac{1}{2}C_{mk}\,\pi\,\rho_w\,g\,H_{max}\,d\,\{\ (1+d/h)\,H/(3L) + 1/(2\pi) + H/(8\pi d)\ \} \qquad (1c)$$

Many versions of Minikin's formula for total force, except that used by BSI (1984), included a factor of 101 replacing $\pi g$, but without qualification on the units. In imperial units this becomes more serious when later authors imply that $\pi g$ = 101 can be used in other units than f.p.s, and have thus propagated the erroneous version of Minikin's formulae ever since! In practice the SPM version of Minikin's method gave so much greater pressures than other formulae that its use for calculations in practical design has been limited. Goda writing on wave force formulae in 1990 summarises the prevalent view on Minikin's method as "can be considered to belong to a group of pressure formulae of historical interest".

Takahashi extended Goda's method to include effects of breaking wave impacts. This was obtained by re-analysing tests of caissons sliding under wave impacts (regular waves), together with data on caisson movements at Sakata Port. The modification is applied by changing the $\alpha_2$ coefficient to be the maximum of $\alpha_2$ or a new impulsive coefficient $\alpha_I$, itself given by coefficients representing the effect of wave height on the mound, and mound shape.

<u>Other methods for impact pressures</u>

Partenscky quoting Oumeraci has used results from the large wave channel at Hannover / Braunschweig (GWK) to suggest that impact pressures of very short durations (0.01 to 0.03s) may be calculated the breaking wave height, and a coefficient $K_L$ given in terms of the air content $a_e$ of the breaking wave.

Blackmore & Hewson conducted field measurements at four 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, $\lambda$, which depends on the roughness of the foreshore. A value of $\lambda$ = 0.3 is recommended for a rough and rocky seabed, and $\lambda$ = 0.5 for a regular seabed. Breaking wave heights are indirectly considered by using shallow water wave velocities calculated from the breaking water depth, $h_b$, and breaking wave height, $H_b$.

More recently, Klammer et al (1996) have developed a method to predict wave impact forces based primarily on solitary waves, but with some (rather tenuous) comparisons with data from random wave tests in the Large Wave Flume at Hannover / Braunschweig.

## 3. DESIGN OF MODEL STUDIES

### 3.1 Test structures and facility

Hydraulic model tests were conducted to measure wave pressures / forces on a range of simple vertical and composite wall configurations, using the Deep Random Wave Flume at Wallingford, which is 52m long and operates with water depths between 0.8m and 1.75m. The flume is configured to reduce reflection of waves from the test section in absorbing side channels. The bathymetry approaching the test section was formed to 1:50. The main caisson was formed as a hollow box with pressure transducers mounted flush with the front face and the underside. The design / construction of the model caissons, the measurement systems, and the test programme, have been described fully by Allsop et al (1996a).

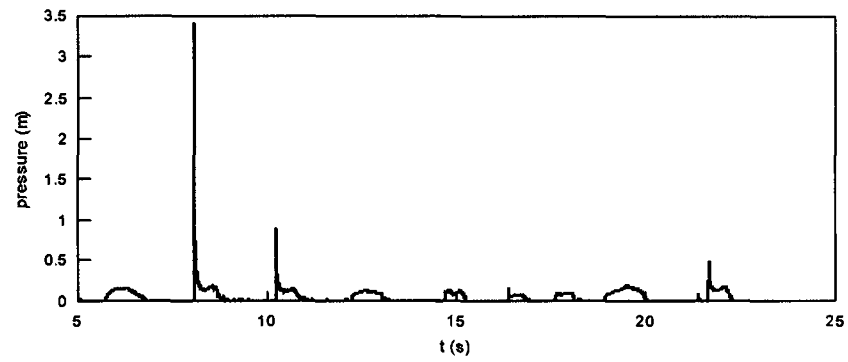


**Figure 4 Wave pressure events from Test 10003 on Structure 1**

The geometric and wave parameters that influence wave forces include: wave heights, $H_{so}$ and $H_{si}$; water depth in front of the structure, $h_s$; crest freeboard, $R_c$; wave steepness, $s_{mo}$; wave length at structure toe, $L_s$; water depth over mound at wall, d; berm height, $h_b$; berm width, $B_b$, and front slope of mound, $\alpha$; and the depth of embedment of caisson into mound, $h_b$-$h_c$. Systematic variations would have required more than 1 year testing, so drastic reductions were made by concentrating on the most important dimensionless parameters, particularly the relative wave height, $H_s/d$, berm length, $B_b/L$, and berm height, $h_b/h_s$.

For the simple wall, the parameters varied were the waves and water depth. The crest level was not changed. For composite walls, the main change was the relative height / depth of the mound in front of the wall, by varying the height of the mound, and the water level. The other changes were to the width of the berm (3 widths) and the front slope angle of the mound (1:1.5 , 1:3 with most tests using 1:2). Eleven structures were tested in this study. Structure 0 was a simple vertical wall. The main composite walls were Structure 1 with a small mound; Structures 2 or 3 with intermediate mounds; and Structures 9 and 10 with large mounds.

Wave steepnesses of $s_{mo}$= 0.02, 0.04 and 0.06 were used for relative wave heights of $H_{si}/h_s$ = 0.1 - 0.6, but restricted to 0.15 - 0.4 for some structures. Up to 5 water levels were used.

Pressures / forces were measured by pressure transducers in the front and lower faces of the caisson. These gave high resolution measurements to about 8m (fresh) water head, and a maximum pressure before damage equivalent to 15m.

The sampling rate was 400Hz, and data were collected from 16 pressure transducers for 500 waves for each test. Oumeraci et al (1994) suggest reduction factors for impact pressures sampled at 400Hz may give under-estimates of maximum pressures by up to 50%, but the reduction for horizontal force is limited to 20%. At these rates, the total impulses are not significantly affected.

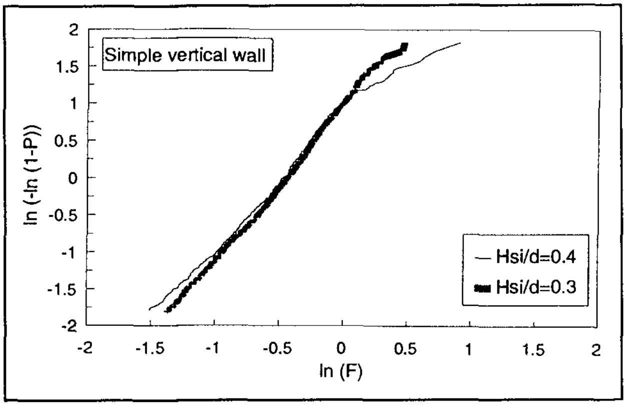


**Figure 5 Weibull probabilities of horizontal forces, pulsating, and (few) impacts**

## 4. RESULTS

In excess of 1 million waves were sampled in 217 tests. Pressures measured by a transducer at static water level are shown in Figure 4 for about 9 waves on Structure 1 with a low rubble mound. This shows some impact events; and others with substantially smaller pressures with much longer rise times, pulsating events. During other tests, severe impacts up to $p=40\rho_w g H_s$ were noted. Previous studies had suggested that severe impacts might be very variable, but repeated tests confirmed that these impacts were repeatable.

**Simple vertical walls**

In the first analysis, horizontal forces (and up-lift where applicable) were calculated for each force event. The statistical distribution of horizontal forces were then plotted on Weibull axes for each test. Examples for simple vertical walls subject to waves of $H_{si}/h_s$=0.3 and 0.4 in Figure 5 show the start of impacts (about 2%) where the highest forces for $H_{si}/h_s$=0.4 start to deviate (upwards) from the linear Weibull line for the pulsating condition for $H_{si}/h_s$=0.3. Overall forces are not greatly increased by these impacts, but $F_{h99.6\%}$ and $F_{h1/250}$ are increased significantly. These increases further as waves approaches the breaking limit for shallow bed slopes around $H_{si}/h_s$ = 0.55 to 0.6.

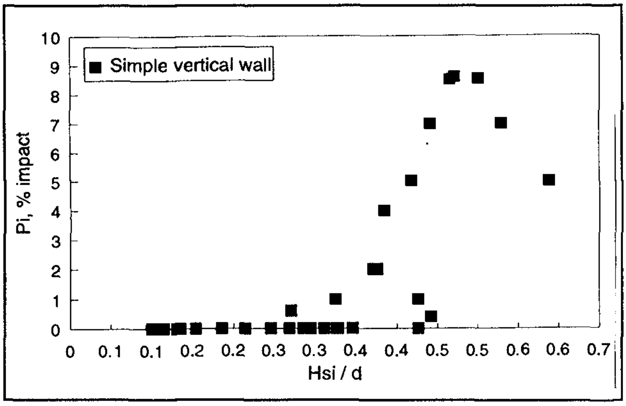


**Figure 6 Influence of $H_{si}/d$ on $P_{i\%}$ for simple walls**

For the simple wall tests, impacts ($P_{i\%}$) are plotted against $H_{si}/d$ (equal to $H_{si}/h_s$ for simple walls) in Figure 6. Impacts start at $H_{si}/d > 0.35$, suggesting this simple limit for onset of impacts. $H_{si}/h_s=0.35$ is lower than the simple rule for wave breaking over shallow bed, but it is reasonable to expect some larger waves to break at conditions below $H_{si}/h_s=0.55$.

These limits identify different types of wave / structure interaction, but do not predict forces. Horizontal forces non-dimensionalised as $F_{h1/250}/\rho_w g d^2$ have been plotted against $H_{si}/d$ in Figure 7. Force predicted by Goda's method are also shown, illustrating relatively good agreement for relatively small waves in the region $H_{si}/d \leq 0.35$, but significant errors for those waves which cause impacts, $H_{si}/d > 0.35$.

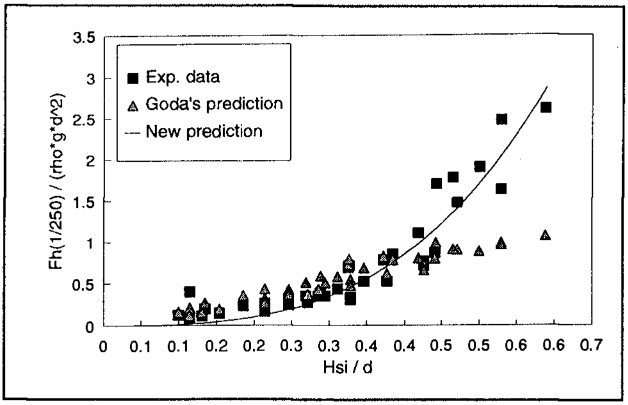


**Figure 7 Dimensionless force against $H_{si}/d$ for simple walls**

**Composite structures**

Responses of composite structures are more complex, being influenced by the height, width and slope of the rubble berm, as well as by relative water depth and wave conditions. The first task was to separate data by the relative berm height, $h_b/h_s$ into "low" and "high" mounds. Low mounds are described by $0.3 < h_b/h_s < 0.6$, and high mounds by $0.6 \leq h_b/h_s < 0.9$. These limits are not themselves of much significance, but give convenient divisions between regions of different response characteristics.

Low mounds, $0.3 < h_b/h_s < 0.6$

For low mounds, the onset of breaking and hence of impact conditions is shifted by the presence of the mound to $H_{si}/d=0.65$, see Figure 8. For higher waves, $0.65 < H_{si}/d \leq 1.3$, the nearness of breaking and effect of the mound combine to increase $P_{i\%}$. The dimensionless forces in Figure 9 fit surprisingly well the simple prediction method in eqn. (2) derived initially for simple walls only.

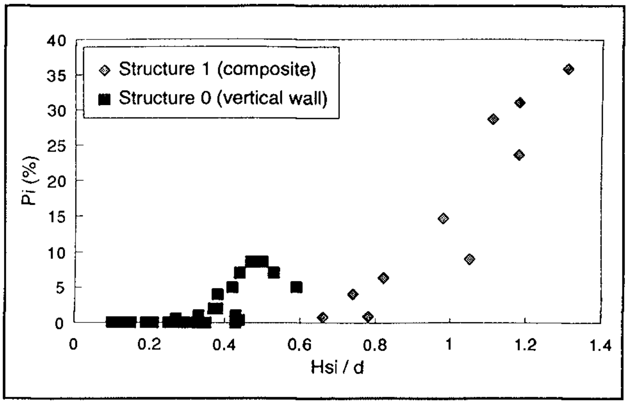


**Figure 8 Influence of $H_{si}/d$ on $P_{i\%}$ for low mounds**

$$F_{h1/250}/(\rho_w g d^2) = 15\,(H_{si}/d)^{3.134} \qquad \text{for } 0.35<H_{si}/d<0.6 \qquad (2)$$

High mounds, $0.6 \le h_b/h_s < 0.9$.

Wave loads are again pulsating for high mounds with smaller waves ($0.3 < H_{si}/d < 0.6$). Here Goda's equations give conservative predictions for horizontal wave forces. Then as wave heights increase, more waves break on the structure, and the situation becomes more complex. For small waves, $0.3 < H_{si}/d < 0.55$, the loading conditions are primarily pulsating, and for larger waves, $0.65 < H_{si}/d < 1.3$, primarily impacts.

Within the last zone examined here, covered by the largest waves tested $0.65 < H_{si}/d < 1.3$, the influence of berm width expressed as $B_{eq}/L_p$ is substantially more important. For short berms, given by $0.08 < B_{eq}/L_p < 0.14$, the waves are still pulsating with few if any impacts, and again Goda's method can be used to estimate wave forces. At the opposite end with long berms given by $B_{eq}/L_p > 0.4$, wave breaking occurs over the berm before the wall, and wave loads on the wall are due to broken waves. Again the use of Goda's method gives a safe estimation of forces, even though the process under broken waves will be rather different.

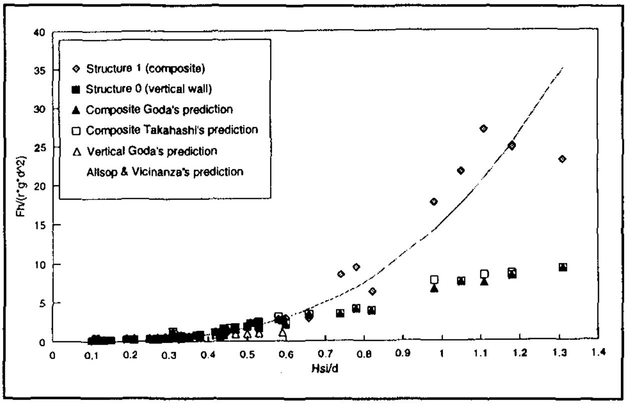


**Figure 9 Dimensionless force against $H_{si}/d$ for low mounds**

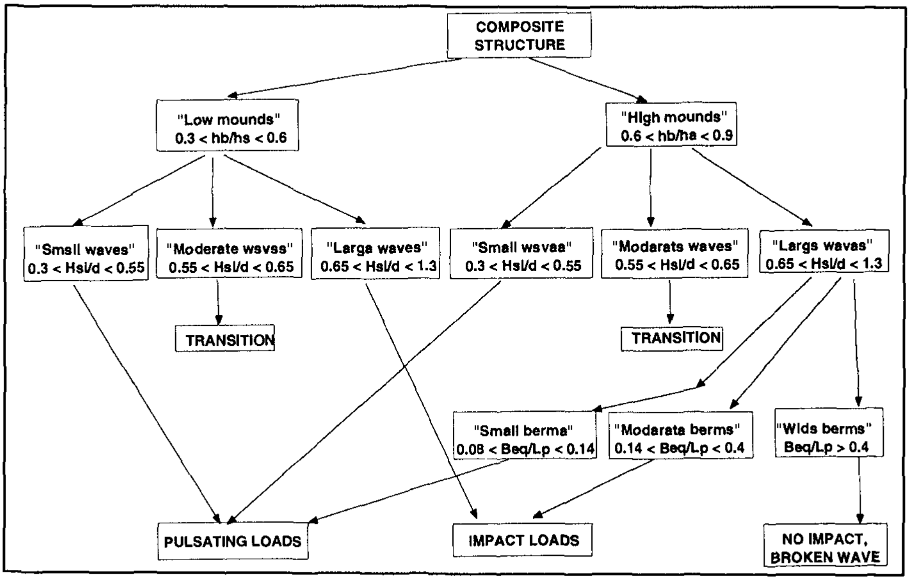


**Figure 10 Decision chart for impacts on simple or composite walls**

The overall picture of the different loading conditions over these regions is summarised in a type of flow chart in Figure 10. This was developed first by Allsop et al (1996c) and refined by Allsop et al (1996a). The parameter regions are divided by the relative berm height $h_b/h_s$, the relative wave height $H_{si}/d$, and the relative berm length $B_{eq}/L_p$. This chart represents a considerable simplification of the overall processes, but renders decisions on the type of wave loading substantially more tractable.

## 5. POSITION AND VARIATIONS OF IMPACTS

The major emphasis so far has been the extreme pressures / forces which determine overall stability of the structure. Data on local pressures and pressure gradients are also needed in any analysis of potential local damage or instability of blockwork.

**Distribution of pressures**
Goda's method assumes that wave pressures are distributed trapezoidally over the front face. For pulsating conditions this assumption is reasonably well-supported, but for impact conditions, agreement is much less good. For $H_{si}/d$=0.4 on the same simple wall, the peak pressures in Figure 11 are much more severe. The peak is greater than predicted, and here is slightly above the water level.

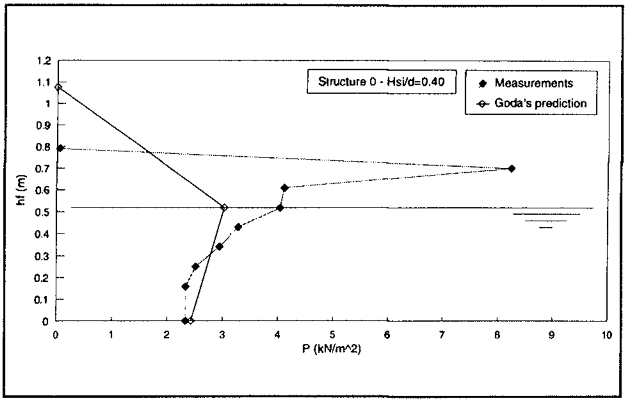


**Figure 11 Vertical distribution of pressures, $H_{si}/d$=0.4, simple wall**

Increasing impacts increase overall forces, but also substantially increases local pressure gradients, illustrated dramatically in Figure 12 where the difference between these tests is in the effective berm width, $B_{eq}$. The most uniformly distributed pressures occur for the simple wall, and for the composite structure with moderate berm, 3. Structures 4 and 7 have only slightly larger berms, yet the local pressures and pressure gradients increase significantly. Here increasing the berm width has initiated the breaking process, giving greater impact forces for greater relative berm width, and dramatically greater peak pressures.

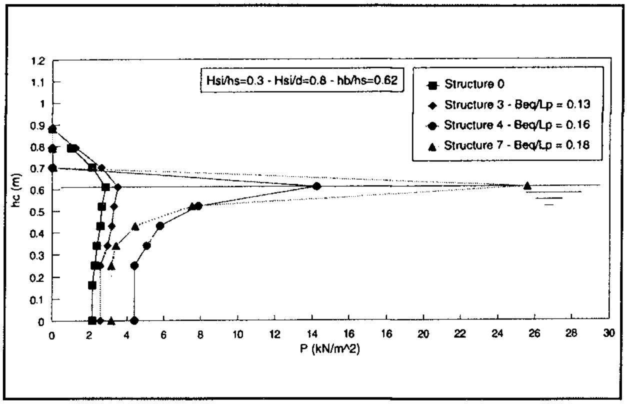


**Figure 12 Effect of berm width on distribution of pressures, high mounds**

**Pressure gradients**

Pulsating wave conditions give relatively low absolute values of wave pressure, so pressure gradients seldom exceed values of dp/dz >1. The situation is however dramatically different for impact conditions, even on simple walls where peak local pressure gradients in these tests varied over dp/dz=2 to 70. These increased slightly for low mounds to dp/dz=5 to 90, and for high mounds to dp/dz=2 to 80. The mean value of these results, the standard deviations (s.d.) and coefficients of variation are summarised below:

| Structure | range | mean (dp/dz) | s.d. (dp/dz) | coef. varn. |
|---|---|---|---|---|
| Vertical | 2 - 70 | 13.2 | 15.9 | 1.19 |
| Low mound | 5 - 90 | 29.5 | 25.9 | 0.879 |
| High mound | 2 - 80 | 21.6 | 17.5 | 0.814 |

For impact waves, the greatest relative local pressure measured in these tests was given by:

$$p_{max} / (\rho_w g H_{si}) \leq 50 \qquad (3a)$$

and the steepest pressure gradient was given by:

$$\max (dp / dz) \leq 90 \qquad (3b)$$

## 6. DISCUSSION ON SCALE EFFECTS

Use of model tests should always include analysis of scale effects, particularly where key responses are influenced by the scale of the tests. Analysis of pressure measurements at laboratory scale here has not explicitly assumed any scale conversion, but use of parameters scaled by Froude implies that pressures measured here can be scaled by Froude unless corrected. Wave impacts in small scale hydraulic model tests are however greater in magnitude, but shorter in duration than their equivalents at full scale in sea water, so simple Froude scaling will over-estimate prototype loads, but under-estimate their durations.

New work discussed by Allsop et al (1996a, b) and Howarth et al (1996) has been used to develop a simple correction method for impact pressures. Measurements of wave impact pressures on concrete armour units on a prototype breakwater have been compared with measurements of equivalent pressures in laboratory tests at 1:32 scale in fresh water. These were used to calculate pressure impulse estimated by peak pressure multiplied by the rise time $\Delta t$. Values of this pressure impulse were compared at the same exceedance levels and show extremely close agreement over the regions of probability of interest, 90-99.9% non-exceedance.

Extreme impact pressures from field and model have then been compared by Allsop et al (1996a, b), see Figure 12, and these show that model results need to be corrected by factors between 0.45 to 0.40 for non-exceedances levels of 90 - 99.9%. It should however be borne in mind that the relatively slow rate of sampling

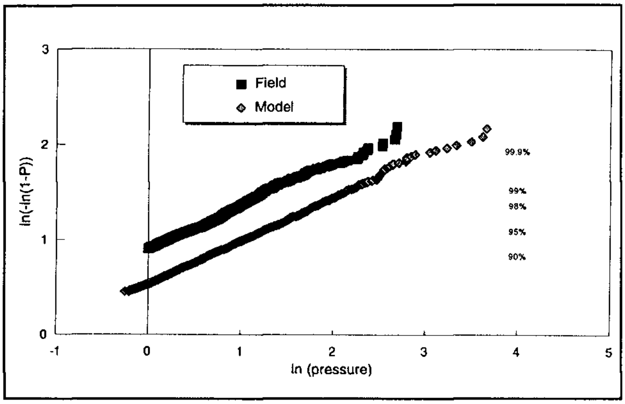


**Figure 13 Weibull probabilities of impact pressures, field and model**

used in the Wallingford / Belfast / Sheffield tests may imply that some peak pressures could have been under-estimated by up to 50%, so any scale correction to reduce predicted pressures should be used with care.


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