Historical Experience of Vertical Breakwaters (in the UK)

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“Italy is often considered as a mother country of vertical breakwaters for harbour protection ... the technology of vertical concrete walls was introduced 2000 years ago by the Roman harbour engineers in contrast with the Greek tradition of rubble mound breakwaters.”

Composite breakwater at Claudius Port (Rome) with concrete superstructure using ship hulls as lost forms
Detached vertical breakwater (blockwork) at the Venetian port at Dubrovnik (circa 1500s).

Armour on the seaward face was added later.
Classic “vertical” breakwaters

The “Cob” breakwater at Lyme Regis, 16th C, Braye & Tatham (1992)

Typical timber frame with rubble hearting, Braye & Tatham citing Shield (1895)
Classic “vertical” breakwaters

Original design for Alderney (c. 1845), showing foundation mound up to just below low water, stone blockwork walls, un-cemented fill
Classic “vertical” breakwaters

Wide mound to break waves before hitting the wall, but high mound can cause (longer) waves to shoal up and break impulsively against the wave wall
Classic “vertical” breakwaters

New Tyne North Pier, 1899
Construction methods, mid 1800s

Construction tools, including placement frames and travelling gantry, diving bell for mound preparation
Construction methods, mid / late 1800s

“Titan” cranes for block placement, here used at Peterhead South Breakwater
Tangier Breakwater, 1661-1684

"The story of the English Occupation of Tangier would be incomplete without some account of the building of the Mole, the greatest engineering work till then attempted by Englishmen."

Tangier, Greate Chest caissons

The revised caisson design, 1677, after Routh (1912)
The “12 January. ….So I went to the Committee, where we spent all this night attending to Sir J. Lawson’s description of Tangier and the place for the Mole\textsuperscript{1} of which he brought a very pretty draught.

\textsuperscript{1} In April, 1663, … the charge for 1 year’s work was £13,000. In March 1665, £36,000 had been spent on it. …. Colonel Norwood reported in 1668 that a breach had been made… which cost a considerable sum to repair.

6 February…where at the Solicitor Generals’ 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 13s the cubic 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.
Dublin Great South Wall

Constructed 1716 – 1786
from Ringsend out to Poolbeg
Dublin Great South Wall

Rennie’s expansion scheme, 1802
Dublin Great South Wall

High water, views from North side

View from the South
Dublin Great South Wall

Indicative cross-section through Great South Wall

MHWS 4.10m LAT

0.22m LAT VARIES

4.22m LAT

4.56m LAT

IN FiLL
Dublin Great South Wall

Borehole photographs, courtesy Jacobs Engineering and Dublin Port Authority
Wave effects on vertical structures

Wave loads and related responses for vertical, battered or composite walls.
Impulsive wave breaking against vertical or battered walls

⇒ high overtopping
+ high velocities
+ intense local pressures
Wave effects on vertical structures

Impulsive loads on vertical wall at Amlwch, small movements, about 1m at breakwater head.

Over-simple wave load formulae.

Ignored research on impulsive wave loadings – but so did everybody else!
“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.”


“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…”

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

Stephenson’s wave force Dynamometer, circa 1845
Wave load and overtopping analysis methods, 1967 - 2010
Wave loads +ve and -ve (graphical method), Goda (1967) and empirical methods Goda (1985, 2000)
Seaward wave loads - vertical walls


Negative forces exceed positive (landward) forces

Sainflou formula: 

\[
p_1' = \rho g (H - h_b) \\

h_b = (\pi H/L) \coth (2\pi b/L)
\]

\[
p_2' = \rho g H / \{\coth (2\pi b/L)\}
\]
Quasi-static or non-impulsive loads, Goda

Wave effects on vertical structures

Takahashi’s impulsive breaking wave pressure coefficient, $\alpha_{I1}$, applied to Goda’s formulae to enhance $\alpha_I$. Takes values of $\alpha_I$ between 0 and 2.

Types of wave loads (PROVERBS)

Dimensionless parameters:
- relative mound height, $h_b^* = \frac{h_b}{h_s}$
- relative wave height, $H_s^* = \frac{H_s}{h_s}$
- relative berm width, $B^* = \frac{B_{eq}}{L_{pi}}$

Vertical breakwater
$h_b^* < 0.3$

Composite breakwater
$0.3 < h_b^* < 0.9$

Crown walls, rubble mound breakwater
$h_b^* > 0.9$

Low mound breakwater
$0.3 < h_b^* < 0.6$

High mound breakwater
$0.6 < h_b^* < 0.9$

Small waves
$0.1 < H_s^* < 0.35$

Large waves
$H_s^*>0.35$

Small waves
$0.2 < H_s^* < 0.6$

Large waves
$0.1 < H_s^* < 0.25$

Small waves
$0.25 < H_s^* < 0.3$

Large waves
$H_s^* > 0.35$

Small waves
$0.1 < H_s^* < 0.2$

Large waves
$0.2 < H_s^* < 0.6$

Wide berm
$B^* > 0.4$

Moderate berm
$0.12 < B^* < 0.4$

Narrow berm
$0.08 < B^* < 0.12$

Pulsating wave loads

Slightly breaking waves

Impact wave loads

Broken waves

\[ h^* = \left(\frac{h}{H_s}\right) \left(\frac{2\pi h}{gT_m^2}\right) \]
Impulsive wave loads from McKenna (1997) PhD tests at Wallingford, see also Allsop et al (1996).
Measurements of dynamic responses

PROVERBS – Lamberti et al (1999), Vol IIb, Ch 3

Motions of the caisson measured by 15 accelerometers

2t sandbag dropped from 5 m

100 tonne tugboat

Votri - Tug-boat excitation

Impulsive response

Final response
Impulsive wave loads - vertical walls

\[ C_p \left( G|P = \zeta \right) = \frac{1}{2} \left\{ 1 - \frac{1 - \zeta (\psi + 1) + G(\psi - 1)}{\sqrt{\Theta^2 - 4 \psi (\psi - 1) \zeta G}} \right\} \]

Dynamic response of a SDOF to pulse excitation

Normalised impact force \( F_{\text{imp}} / F_{qs+1/250} \)

Normalised rise time \( t_r / T_m \)

Permeabilities of blockwork and fill

a) open / permeable, $k = \text{high, constant (approx.)}$
b) grouted / sealed, $k = \text{low, constant (approx.)}$
c1) open both faces, gradual internal permeabilities
c2) closed front face, stepped permeability gradients
c3) closed both faces, stepped permeability gradients

Permeabilities of blockwork and fill

a) open / permeable, $k = \text{high and constant (approx.)}$

After Bruce et al, ICCE (2000)
Permeabilities of blockwork and fill

b) grouted / sealed, \( k = \text{low, constant (approx.)} \)

After Bruce et al, ICCE (2000)
c2) stepped permeability gradients, closed front face

After Bruce et al, ICCE (2000)
Permeabilities of blockwork and fill

c3) stepped permeability gradients, closed both faces

After Bruce et al, ICCE (2000)
Port Logan, Rhinns of Galloway. Failure of close fitting blockwork armour (low permeability) over ungrouted rock fill.
Wave loads on vertical structures


[Photos courtesy Dr Gerald Muller, Univ. Southampton.]

Stress flow within a blockwork wall simulating load transfer from a parapet wall onto a battered face, studies by Muller et al (2002).

[Photo courtesy Dr Gerald Muller, University of Southampton.]

Dover Breakwaters (Admiralty and Western)

1880-1900
Dover harbour, 1880 - 1900
Dover breakwater, showing concrete blocks, use of bag joggle jointing, no foundation mound. Required large plant and divers.
Dover harbour, 1880 - 1900
Dover harbour, 1880 - 1900
Admiralty breakwater, Alderney
High mound causes (longer) waves to shoal up and break impulsively against the upper wave wall.
Alderney Breakwater under storms
Alderney Breakwater – damage repair
Admiralty breakwater, Alderney
Alderney – loss of mound material

Mound losing 3000 – 6000m³ per year, rate of loss tending to accelerate 1990 - 1993

Changes to the mound volume, 1960-1990
Alderney – wave velocities / pressures
Alderney – pressures within the mound
Alderney – wave loads

HRS model tests (circa 1965) studying movement of the mound material.
Alderney – wave loads

Wave loads (per m run) calculated using Goda + Takahashi for 1:50 or 1:20 year waves of $H_s = 8.4\text{m}$ (or $7.4\text{m}$).

NB This does not calculate spatially limited impact pressures which could be 5-20 times greater.
Alderney – the “dip” measurements
More recently, Hitachinake

Pre-cast concrete caissons for Hitachinake port constructed on land, slid into launching dock, and floated out to position.
Even more recently, Costa Azul

Pre-cast concrete caisson breakwater for Costa Azul in Mexico, constructed in (temporary) dry dock, towed out to position.
and finally, Mutriku, Basque country
Expansion of Mutriku harbour, new outer breakwater with 16 Oscillating Water Column chambers in vertical wall section
Early proposal for Oscillating Water Column chambers formed in precast caissons
Mutriku, Basque country

Contract awarded for Oscillating Water Column chambers constructed using precast “ring” sections to form vertical wall.
Construction of OWC breakwater using precast “ring” sections to form wall and chambers
Mutriku, Basque country
Mutriku, Basque country
Wave loads (per m run) calculated using Goda + Takahashi for 1:10 bed slope, design wave of $H_s = 5$ m, and extreme (revised?) condition of $H_s = 8$ m.
Conclusions, and further remarks

- Essential to understand the wave loading regime
- Understand the composition of the structure when built, and now, and in the future
- Dynamic analysis essential for all structure experiencing impulsive loads
- Appropriate permeability gradients, avoid reverse or steep hydraulic gradients
- Alderney breakwater – see 1991 paper.
- Where next?
Design / analysis of vertical breakwaters

a) Design / construction of vertical breakwaters was primarily based on experience for much of the 1800s (and 1900s)

b) Changing technology altered construction methods, in turn changing key design decisions;

c) Momentum based (non-impulsive) wave loads are well predicted by the semi-empirical methods of Goda (1985, 2000)

d) Impulsive loads can exceed non-impulsive loads by 5-50 times, but are limited spatially and in duration
Design / analysis of vertical breakwaters

e) Analysis of impulsive load effects must use dynamic methods

f) Performance of blockwork systems depends critically on the permeabilities of outer and inner “layers”

g) Stability of individual blocks depends on impulsive pressures, internal transmission of short duration pressures, and structural support from adjoining blocks and the foundation
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