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Failure analysis of historic vertical breakwaters, part 1: Wick

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Many historic breakwaters failed early in their life, leaving little information by which to analyse or understand their failures. As part of a wider analysis of 'old breakwaters', the first author has analysed the 'stability' of example breakwaters using analytical methods developed over the past 20 years with co-researchers. This analysis is illustrated by three case studies, the first covered in this paper and the second two in the companion paper: Wick (designed by Thomas Stevenson, failed 1870–1877)—Alderney (damaged even during construction, lost its outer length 1865–1889)—and 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 (Probabilistic Design Tools for Vertical Breakwaters) project have been used to explore the incidence of wave impact loads, the main momentum loads and the impulsive loads. Factors of safety against sliding and/or overturning have been determined for each example over a range of representative wave conditions and compared with reality.

Notation

B^*	dimensionless berm width, B_{eq}/L_p
B_{eq}	effective width of mound/berm in front of the wall, taken at one-half mound height (m)
C^*	relative reflection coefficient (=(1 − C_r)/(1 + C_r))
C_r	reflection coefficient
F_h	horizontal wave force per unit length (kN/m)
H_s	significant wave height (m)
H_s^*	relative wave height, H_{si}/h_s
H_{sb}	significant wave height (broken)
H_{si}	incident significant wave height (having taken account of all wave transformations) (m)
H_{ss}	significant wave height (shoaled) (m)
h	water depth, varies with water level (m)
h_b	depth over the mound or berm (m)
h_b^*	relative berm depth, h_b/h_s
h_s, d_s	water depth at the structure (m)
K_r	coefficient of refraction
K_s	coefficient of shoaling
L	wavelength (m)
L_p	peak period wavelength (m)
m	bed slope (1:x)
P_b	proportion (or percentage (%)) of waves breaking
p_{av}	average wave pressure (kN/m^2)
T	wave period (s)
T_m	mean wave period (s)
T_p	peak wave period (s)
α	front face slope (of foundation mound) (1:x)
α_1, α_2, α_3	coefficients in Goda's formulae
β	angle of wave obliquity (°)
γ	wave breaker ratio, H_{sb}/h_s

Note 1: Most definitions are given in full in *The Rock Manual* (Ciria *et al.*, 2007), ISO/CD 21650 (ISO, 2007), or the PROVERBS (*Probabilistic Design Tools for Vertical Breakwaters*) book by Oumeraci *et al.* (2001).

Note 2: Some subscripts are condition sensitive, so subscript 'b' indicates breaking but only when dealing with wave conditions – for example, as in H_{sb} – otherwise, 'b' may denote a berm or mound.

Note 3: Proportion or % is defined at the point of use.

1. Introduction

The trade and defence of the UK have depended critically on its harbours. Indeed, trade of coal and foodstuffs around Scotland and coastal regions was nigh impossible without maritime transport. On exposed coastlines, the harbours required to support this trade are formed by man-made breakwaters. In the absence of robust design methods, many historic breakwaters failed early in their life, leaving few data to analyse or understand their failures. As part of a wider analysis of 'old breakwaters' (Allsop, 2020; Allsop, *Old British Breakwaters – How Has History Influenced Their Survival?*, PhD thesis in preparation, University of Edinburgh), this paper describes the stability of the example breakwater at Wick using analytical methods developed over the past 20 years. This analysis is extended by two further case studies in the companion paper (Allsop and Bruce, 2020), so covering together

- Wick (designed by Thomas Stevenson, failed 1870–1877)
- Alderney (damaged even during construction, lost its outer length 1865–1889)
- Dover (still shows high stability after 110 years).

For each study breakwater, representative cross-sections and approach bathymetry have been derived from historical records. Representative wave conditions are transformed to the breakwater toes, including depth-limiting and impulsive breaking effects. Empirical formulae developed during and since the PROVERBS (Probabilistic Design Tools for Vertical Breakwaters) project (Oumeraci *et al*., 2001) have been used to identify the occurrence of wave impact loads, the main momentum loads and the impulsive loads. Factors of safety (FOSs) against sliding and/or overturning have been determined for each example and then compared with reality. The methods described in this paper are applied to the failure of the Wick breakwater here and to the breakwaters at Alderney and Dover in the companion paper (part 2) as reported by Allsop and Bruce (2020).

2. Analysis methods

The intention of the stability analysis here is to calculate FOSs against the sliding or overturning of representative sections. The primary drivers for these responses are wave loads, so the first steps in the analysis are to determine appropriate wave conditions at the structure, from which are calculated the degree of wave breaking onto the breakwater wall. The disturbing loads are then contrasted with the structure weight and frictional resistance. The methods used here were first presented by Allsop (2000)

2.1 Wave analysis

Waves have been shoaled from offshore conditions using classical shoaling equations (checked against the graphical method in the *Shore Protection Manual* (SPM) (USACE, 1977)). Where wave attack is closely normal to the seabed contours, the effect of refraction on changing wave heights will be relatively small. As waves run into shallower water, the wave crest becomes steeper, and taller, and the trough flattens. Generally, the wave height increases until it reaches a limit and then the wave breaks.

In the analysis here, the shoaling coefficient is calculated by using a simple linear theory; see, for example, the US Army Corps of Engineers' (USACE) SPM or the recent *Coastal Engineering Manual*. Shoaled wave heights are then checked for depth-limited breaking using the depth-limiting equations by Owen (1980), which provide the upper limit to the wave height H_{sb} under wave breaking relative to the water depth h_s or d_s, $\gamma = H_{sb}/h_s$. The important aspect of Owen's simple curves is that they include the effects of steep bed slopes (m) often omitted (or oversimplified) by other methods (Figure 1). This can be critical for breakwaters formed on natural shoals and/or rubble mounds.

2.2 Occurrence of wave forces

In PROVERBS (Oumeraci *et al*., 2001), the researchers developed the parameter map (Figure 2) based on physical model measurements (Allsop *et al*., 1995, 1996). These estimate the occurrence of impulsive loads by assessing three dimensionless parameters

- $h_b^* = h_b/h_s$: relative 'mound' height to total water depth
- $H_s^* = H_{si}/h_s$: incident wave height relative to water depth
- $B^* = B_{eq}/L_p$: mound width relative to wavelength.

A simple model of wave breaking was developed within PROVERBS by Calabrese to give estimates of the proportion or percentage of wave impacts on vertical/composite walls. Breaking occurs when, at the structure, the incident wave height with an exceedance probability of 0·4% ($H_{99\cdot6}$) is higher than a critical breaker height H_{bc}, defined as the transition wave height between breaking and non-breaking in front of the structure. The equivalent berm width, B_{eq}, is estimated at one-half berm height and the peak wavelength L_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), $0{\cdot}07 < h_s/L_p < 0{\cdot}25$

1. $$H_{bc} = (0{\cdot}1025 + 0{\cdot}0217C^*)L_p \tanh\left(2\pi k_b h_s/L_p\right)$$

where

$$k_b = 0{\cdot}0076\left(B_{eq}/d\right)^2 - 0{\cdot}1402\left(B_{eq}/d\right) + 1$$

2. $$\text{for } 0 \leq B_{eq}/d < 10$$

and

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

Reflections C_r may be estimated using the guidance by Allsop (1995)

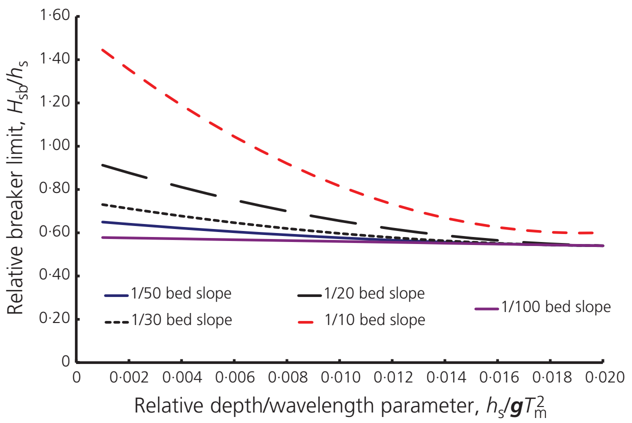

Figure 1. Depth-limiting breaking curves (Owen, 1980)

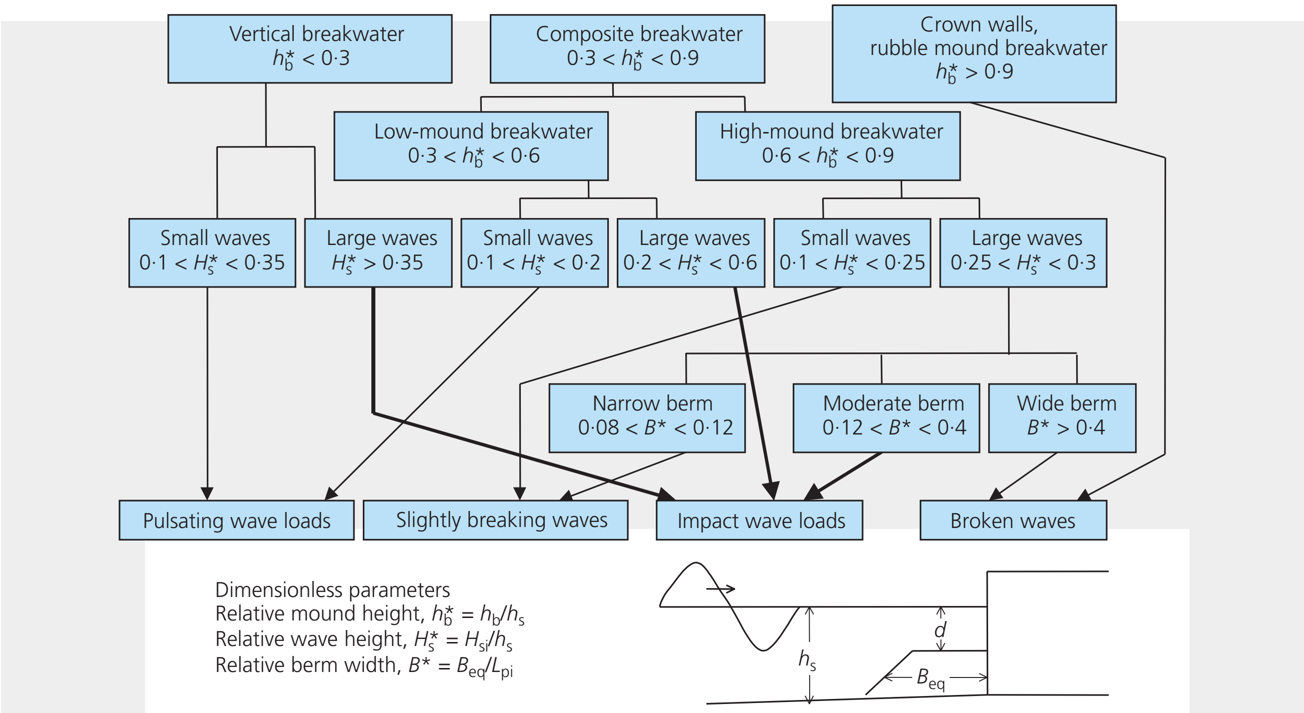

Figure 2. PROVERBS parameter map for wave impacts on vertical walls (Oumeraci *et al.*, 2001)

for simple vertical walls and small mounds:

4. $C_r = 0{\cdot}95$

for low-crest walls ($0{\cdot}5 < R_c/H_{si} < 1{\cdot}0$):

5. $C_r = 0{\cdot}8 + 0{\cdot}1R_c/H_{si}$

for composite walls, large mounds, heavy breaking:

6. $C_r = 0{\cdot}5 - 0{\cdot}7$

The uncertainties in predicting breaking suggest a conservative approach assuming $C_r = 1$, so $C^* = 0$. H_{bc} reduces to

7. $H_{bc} = 0{\cdot}1025L_p \tanh(2\pi k_b h_s/L_p)$

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

- $H_{si}/H_{bc} \leq 0{\cdot}6$: no evident breaking and wave load is pulsating
- $0{\cdot}6 < H_{si}/H_{bc} < 1{\cdot}2$: wave breaking occurs and may give impacts
- $H_{si}/H_{bc} \geq 1{\cdot}2$: heavy breaking may give broken loads.

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

8. $P_{b\,\%} = \exp\left[-2(H_{bc}/H_{si})^2\right] \times 100\,\%$

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

9. $$P_{i\,\%} = \left\{\exp\left[-2(H_{bc}/H_{si})^2\right] - 0{\cdot}58\exp\left[-1{\cdot}93(H_{bc}/H_{si})^2\right]\right\} \times 100\,\%$$

Values of $P_{i\%}$ may then be used to reappraise the likely loading case

- $P_{i\%} < 2\%$: little breaking and wave loads are primarily pulsating
- $2\% < P_{i\%} < 10\%$: breaking waves give impacts
- $P_{i\%} > 10\%$: heavy breaking gives impacts or broken loads.

2.3 Wave 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. The method was calibrated against laboratory tests and analysis of historic failures. Goda emphasised that it does not predict wave pressures, even though in practice, many researchers have found it to 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.

However, before considering Goda's method, it is useful to review the very simple methods by Ito and/or Hiroi; see the publications by Ito (1971) and Goda (1985). Hiroi's formula calculates a uniform wave pressure (p_{av}) on the front face up to $1{\cdot}25H$ above the still water level

10. $$p_{av} = 1{\cdot}5\rho_w \boldsymbol{g} H$$

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 $d/H_s > 2$. The method by Sainflou (1928) generally gives $p_{av} = 0{\cdot}8$ to $1{\cdot}0\rho_w gH$, lower than Hiroi's.

Ito's method gives 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}/h_t, where h_t = depth at the toe of the wall

for $H_{max}/h_t < 1$:

11. $$p_{av} = 0{\cdot}7\rho_w \boldsymbol{g} H_{max}$$

for $H_{max}/h_t > 1$:

12. $$p_{av} = \rho_w \boldsymbol{g} H_{max}(0{\cdot}15 + 0{\cdot}55H_{max}/h_t)$$

The more complete (and widely accepted) prediction method for wave loads on vertical walls by Goda (1974, 1985) represents wave pressures on the wall by a trapezoidal distribution, reducing from p_1 at the static water level (SWL) to p_3 at the caisson base (Figure 3). Above SWL, pressures reduce to zero at the notional run-up point given by η^* above SWL. Underneath the caisson, uplift pressures at the seaward edge (p_u) are determined by a separate expression. Uplift pressures are distributed triangularly from the seaward edge to zero at the rear heel.

The main parameters determined in Goda's method are

13. $$\eta^* = 0{\cdot}75(1 + \cos\beta)H_{max}$$

14. $$p_1 = 0{\cdot}5(1 + \cos\beta)\left(\alpha_1 + \alpha_2 \cos^2\beta\right)\rho_w \boldsymbol{g} H_{max}$$

15. $$p_2 = p_1/[\cos\mathrm{h}(2\pi h/L)]$$

16. $$p_3 = \alpha_3 p_1$$

17. $$p_u = 0{\cdot}5(1 + \cos\beta)(\alpha_1\alpha_3)\rho_w \boldsymbol{g} H_{max}$$

Coefficients α_1, α_2 and α_3 are determined from

18. $$\alpha_1 = 0{\cdot}6 + 0{\cdot}5[(4\pi h/L)/\sinh(4\pi h/L)]^2$$

19. $$\alpha_2 = \min\left\{[(h_b - d)/3h_b](H_{max}/d)^2,\ 2d/H_{max}\right\}$$

20. $$\alpha_3 = 1 - \left(h'/h\right)[1 - 1/\cos\mathrm{h}(2\pi h/L)]$$

where η^* is the maximum elevation above SWL to which pressure could be exerted (taken by Goda as $\eta^* = 1{\cdot}5H_{max}$ for normal wave incidence) and β is the angle of wave obliquity. The wave height $H_{max} = 1{\cdot}8H_s$ seawards 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 $5H_s$ seawards of the structure.

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

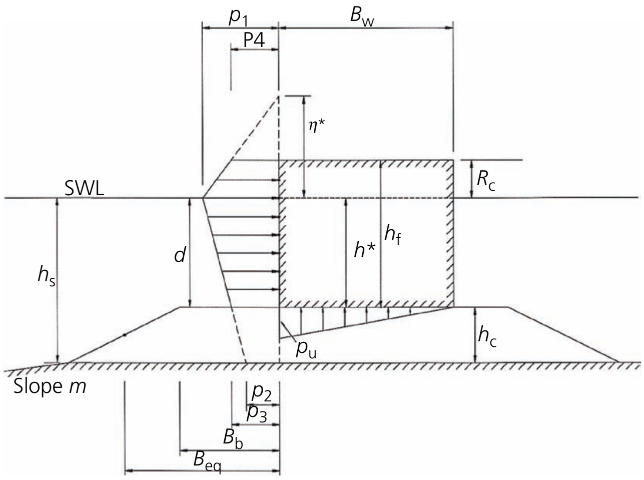

Figure 3. Notation used for Goda's equations for wave force

front face. Total uplift force is calculated by integrating from $p = p_u$ at the front edge to $p = 0$ at the rearward edge, giving a total uplift force $F_u = 0{\cdot}5 p_u B_c$.

2.4 Impulsive forces

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

21. $$F_{h,imp} = C_r^{1{\cdot}65} \rho \boldsymbol{g} H_{si} L_s \left[1 - \left(h_{bimp} - d\right)/d\right]$$

22. $$F_{h,qs} = 4{\cdot}8 \rho \boldsymbol{g} H_{si}^2$$

Taken alone, impulsive forces are, however, of little significance, as their effect 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

23. $$F_{imp} = a t_r^b \quad \text{where } a = 7 \text{ and } b = -0{\cdot}6$$

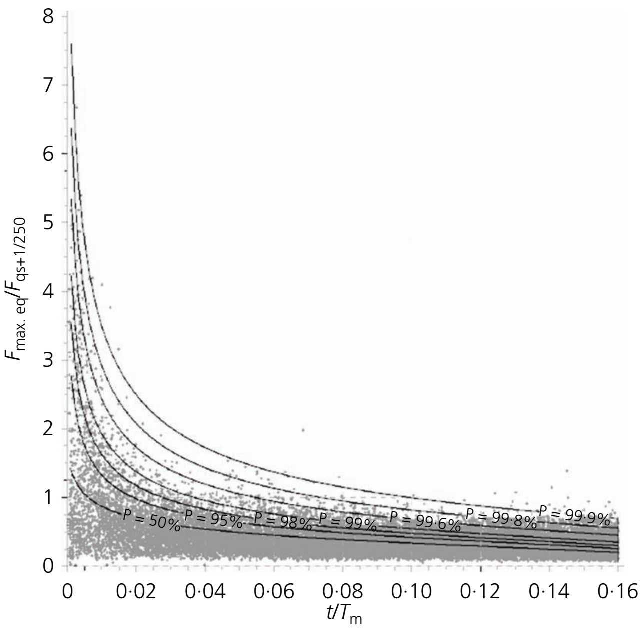

Figure 4. Relative impulsive force plotted against relative rise time (after Cuomo *et al*. (2011))

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 4).

3. Wick

3.1 Stevenson's breakwater

Wick harbour in north-east Scotland 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 in 1825–1834. Even so, by 1857 more capacity was needed, so the British Fishery Society proposed a new breakwater (Figure 5). Plans, sections 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, and the £62 000 loan was approved by A. M. Rendel as engineer to Public Works Loan Commission; see the paper by Paxton (2009). Construction began in April 1863 to a planned length of 460 m.

The design by Stevenson (1874) was a rubble mound to −5·5 metres above low water (mLW) following the Crane Rocks surmounted by block walls, filled between by rubble, with a superstructure width of up to 16 m (Figures 6 and 7). 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 above HW used Roman at the start of the works and then Portland cement mortar. (In his book, Stevenson (1874) fails to define 'Roman cement', so it is assumed to be portmanteau for any pozzolanic cementitious material.) Paxton (2009) claims that the depth to which blocks were taken at −5·5 mLW was 50% deeper than 'the accepted norm' to

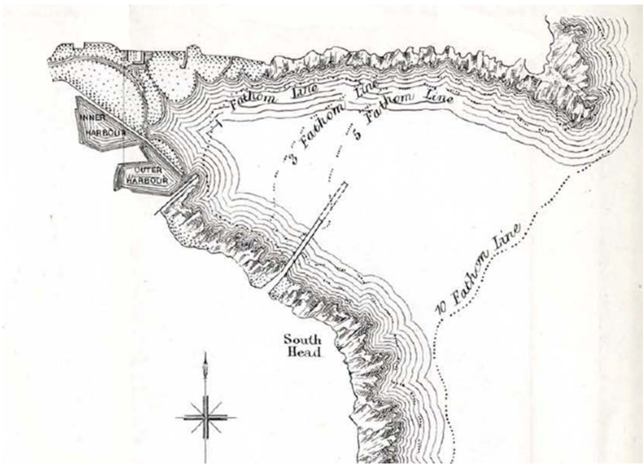

Figure 5. Location of the proposed breakwater (after Vernon-Harcourt (1885)). 1 fathom = 1·83 m

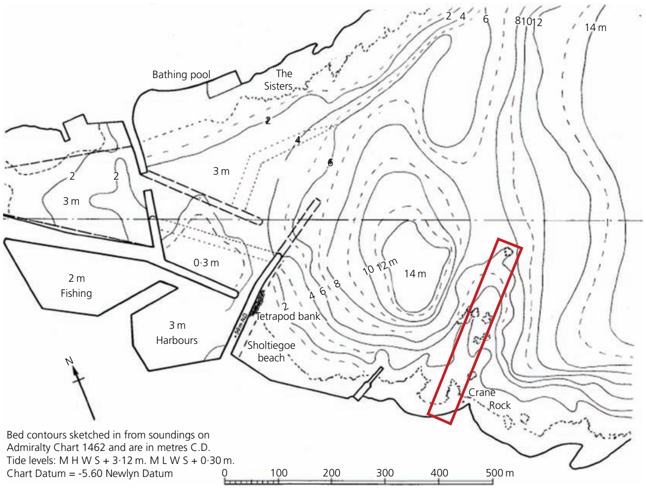

Figure 6. Wick Bay bathymetry (from *Hydraulics Research Station Report EX706* (HRS, 1975))

avoid impulsive breaking over the mound and/or to reduce the likelihood of movement of foundation material.

By October 1867, the rubble had reached 326 m, breakwater walls to 250 m. By September 1868, the completed breakwater had reached 320 m, but in October the outer 75 m was demolished with the wall down to −4·6 mLW. Then, in February 1870, the outer 116 m was knocked over to −1·8 mLW (above the wall foundation level) in a storm estimated at $H_{max} \approx 13$ m and $H_s \approx 7$ m.

In the summer of 1870, the outer 55 m length was rebuilt (to 260 m instead of the original 480 m). 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.

In February 1870, the facing stones were shattered by an ‘unparalleled’ sea, waves about 9 m and spray about 60 m high. Then, in December 1872, the replacement composite end (1372 t) was demolished down to −3 mLW, being ‘slewed round by successive strokes until removed’ (Paxton, 2009: p. 32). Paxton (quoting an unpublished Institution of Civil Engineers (ICE) paper by Doull) notes that ‘the rubble base (at −5 mLW) is said to be not much disturbed’ (Paxton, 2009: p. 37). A further 46 m ‘of solid structure set in cement’ were then destroyed with water 8–9 m deep passing over the parapet. After further damage in January 1877 when a 2642 t 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 have been imparted by overtopping downfall pressures; see the publications by Bruce *et al*. (2001) and Wolters *et al*. (2005). The safety factor analysis will start with momentum-driven loads, as discussed in Section 2, later extending to include impulsive loads.

Figure 7. Location of proposed Stevenson breakwater (after Paxton (2009)). 1 fathom = 0·00183 km

3.2 Water levels and wave conditions

The general bathymetry of Wick Bay (Figure 6 adapted from Admiralty chart 1462 by the Hydraulics Research Station) shows the shoal of the Crane Rocks along which the Stevenson breakwater was built. Paxton (2009) shows the layout along the 5-fathom (0·00914 km) contour (Figure 7), together with an aborted ‘harbour of refuge’ breakwater following the 7-fathom (0·012 km) contour. A dashed box has been overlain on Figure 6, indicating the 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 given as 3·8 m. The main tidal levels (relative to lowest astronomical tide = chart datum (CD)) are listed by Paxton (2009) from Admiralty Tide Tables

- mean high water springs (MHWS): +3·5 metres above chart datum (mCD)
- mean high water neaps: +2·8 mCD
- mean sea level: +2·0 mCD
- mean low water neaps: +1·4 mCD
- mean low water springs: +0·7 mCD.

Paxton (2009: p. 33) suggests that Stevenson ‘would have expected waves of (H_s =) 7–9 m’. For their model tests, HRS (1975) derived a 1:1 year condition as H_{max} = 12 m (H_s = 6·7 m) and a 1:50 year condition as H_{max} = 18 m (H_s = 10 m). Wave periods used were T = 14 s for the longest fetches down to T = 7 s for frequently occurring conditions. Stability calculations here have used H_s = 8 m and H_s = 10 m.

To derive incident conditions, offshore waves must be transformed (shoaling and breaking) over the past 50–100 m.

Three representative sections have been taken across the line of Stevenson's breakwater mound (dashed box in Figure 6), approximately normal to the −10 mCD contour, and taken at chainages of 100 m (A), 180 m (B) and 250 m (C) from the shoreline. Seabed slopes here average 1:10–1:20 at about 50 m from the breakwater toe, at −3 mCD for section A; −5·4 mCD for section B; and −7 mCD for section C.

These calculations were run for the (nominal) water level of MHWS = 3·5 mCD. Tidal ranges at Wick are small, but during any large storm, it is probable that the attendant surge will elevate water levels, so an initial guess at +3·5 mCD seems reasonable.

Three alternative wave periods (T_p = 10, 12 and 14 s), two nominal wave heights of H_s = 8 and 10 m, and a water level of +3·5 mCD have been used to estimate wave heights for each of mound section lines: A, B and C. In the first stage, classic linear shoaling has been used to estimate shoaling coefficients, K_s (column 8 in Table 1). Those values of K_s have been checked (with good agreement) using the SPM graphical method. Shoaled (significant) wave heights in column 9 (H_{ss}) are then tested in column 10 for depth-limited breaking using the methods by Owen (1980), 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 1.

The influence of these steep bed slopes is immediately seen. Shoaling is substantial (10–20% increase) for longer wave periods (T_p = 12 or 14 s). On the steep bed slopes, breaking limits are substantially higher than they would be for shallow slopes. Working seawards, 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, 180 and 250 m from the shoreline, respectively).

Longer wave periods increase shoaling and depth-limited heights. Most focus here will be given to T_p = 12 s for H_s = 10 m and to T_p = 10 s for H_s = 8 m.

3.3 Breakwater section

Paxton (2009) shows two wall sections (Figure 8) at about 250–270 m from the shoreline, but they require interpretation before calculating wave loads. Firstly, levels need to be related to CD. It is assumed here that high water of ordinary spring tides = MHWS = +3·5 mCD. The walkway deck is at +6·4 mCD, the parapet crest at +9·9 mCD, the wall toe on the mound at −3·2 mCD and the seabed at about −8 mCD. The structure width at low water (LW) is about 13 m, and the mean parapet width 2·7 m.

At foundation at −3·2 mCD, the breakwater wall is 9·6 m high and 13 m wide. The parapet adds a further 3·5 m height and 2·7 m width. The parapet density is assumed at 2·6 t/m^3, being fitted stone blocks and concrete. The lower section will be less dense as the fill may be (say) 1·8 t/m^3. Taken overall, the dry weight of this section will be 270 t/m. (The submerged weight will be less, depending on the water level.)

The toe levels are slightly at variance with Paxton's 'failure record', which has the wall founded at −3·6 mLW in October 1864, so probably perhaps landwards of this paper's analysis section C. For October 1867, Paxton gives the wall founded at −5·5 mLW at 250 m out, probably close to this paper's analysis section C (taken at 250 m out from the shoreline).

For analysis of wave loads at C, two alternative wall sections were originally considered, based on dimensions derived from Figure 8 but with the foundation either at −3·2 mCD (section C1) or at −5·5 mCD (section C2). The (dry) weight of section C2 therefore increases by 54–303 t/m.

Table 1. Results of wave transformation calculations

Mound section	H_s: m	T_p: s	Wave steepness	Seabed: mCD	Water depth: m	Bed (1:*x*)	K_s	H_{ss}: m	H_{sb}: m	H_{si}: m
A	10	14	0·033	–3·0	6·5	10	1·2	12·0	8·2	8·2
B	10	14	0·033	–5·4	8·9	10	1·1	11·2	10·6	10·6
C	10	14	0·033	–7·0	10·5	10	1·1	10·8	12·0	10·8
A	10	12	0·044	–3·0	6·5	10	1·1	11·3	7·6	7·6
B	10	12	0·044	–5·4	8·9	10	1·1	10·5	9·5	9·5
C	10	12	0·044	–7·0	10·5	10	1·0	10·2	10·5	10·2
A	10	10	0·064	–3·0	6·5	10	1·1	10·5	6·6	6·6
B	10	10	0·064	–5·4	8·9	10	1·0	9·9	7·9	7·9
C	10	10	0·064	–7·0	10·5	10	1·0	9·6	8·5	8·5
A	8	12	0·036	–3·0	6·5	10	1·1	9·0	7·6	7·6
B	8	12	0·036	–5·4	8·9	10	1·1	8·4	9·5	8·4
C	8	12	0·036	–7·0	10·5	10	1·0	8·2	10·5	8·2
A	8	10	0·051	–3·0	6·5	10	1·1	8·4	6·6	6·6
B	8	10	0·051	–5·4	8·9	10	1·0	7·9	7·9	7·9
C	8	10	0·051	–7·0	10·5	10	1·0	7·7	8·5	7·7

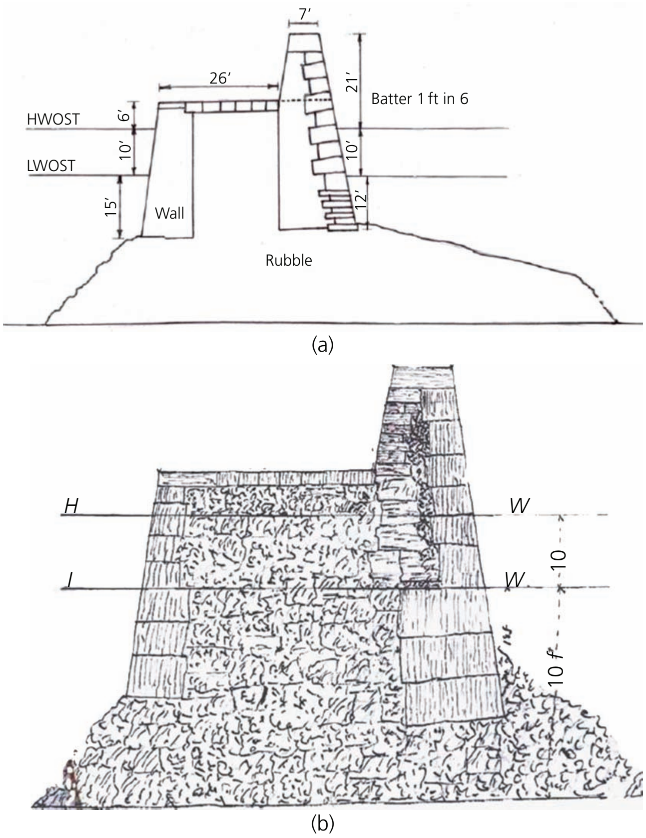

Figure 8. Cross-sections of Stevenson's breakwater (after Paxton (2009)). HWOST, high water of ordinary spring tides; LWOST, low water ordinary spring tides. 1′ = 1 ft = 0·305 m

3.4 Occurrence of impulsive forces

For initial calculations on Stevenson's breakwater, starting with position C at the outer end, local depths reach h_s = 13 m, and over the foundation mound, d_b = 6·7 m. So h_b^* = 0·5, classed as a 'low mound'.

As approach slopes are steep at 1:10 to 20, it is not surprising that relative wave heights are high at H_s^* = 0·60–0·75, giving 'large waves', even for $H_s \approx$ 8 m. The foundation mound is relatively small, $B_{eq} \approx$ 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 ('impulsive wave load' box in Figure 2), particularly over the outer end where waves are shoaled by local bathymetry.

In these calculations, H_{bc} is a fictional rather than measured parameter and may differ significantly from breaking significant wave heights determined by other methods.

The results of these calculations are summarised in Table 2. These confirm the view formed earlier, even at H_s = 8 m (and certainly at H_s = 10 m), that wave breaking onto the breakwater will have been substantial with breaking waves of order 15–30%.

3.5 Wave loads

Wave loads on the wall are influenced significantly by local wave heights and by the geometry, so loads do not always respond in a simple way.

An initial conclusion of the load calculations in Table 3 is that Ito's simple method gives total forces that are higher than those of Goda's more complicated method and probably therefore more conservative. (This may be partially artificial, as Ito pressures are often higher than those for Goda but are applied only over the submerged wall height, so not including the wall to R_c or η* as applied in Goda's method.)

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

3.6 FOS analysis

The simplest stability analysis is to compare horizontal and vertical loads against 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 8) can be divided into three parts

- rubble mound, approximately 34 m wide at the base and from −9·6 mCD up to the wall toe −3·2 mCD, so about 6·4 m high
- main wall section founded at −3·2 mCD (ignoring the ~1 m lower on the rear side) and rising to the walkway at +6·4 mCD, so about 9·6 m high; at mid-height, the wall is about 13 m wide (front to back), so occupies about 125 m^2

Table 2. Example wave breaking calculations

Section	H_s: m	T_p: s	Bed level: mCD	Water depth: m	H_{si}: m	H_{bc}: m	H_{si}/H_{bc}	P_i: %	P_b: %
A	10	12	−3·0	6·7	7·6	2·84	2·7	76	31
B	10	12	−5·4	9·5	10·5	4·47	2·3	70	29
C	10	12	−7·0	11·5	10·2	5·61	1·8	55	22
A	8	10	−3·0	6·7	6·6	2·80	2·4	70	29
B	8	10	−5·4	9·5	7·9	4·37	1·8	54	22
C	8	10	−7·0	11·5	7·7	5·43	1·4	37	15

Table 3. Results of wave load calculations, Ito and Goda

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

- parapet wall to +9·9 mCD, so about 3·5 m high, and of an average width of 2·7 m, so occupying about 9·5 m^2.

Loose fill will have a density of about 1·7 t/m^3, but for fitted masonry, this will be closer to 2·7 t/m^3. In the analysis of example breakwaters by Allsop *et al*. (2017), the average ratios of block to fill were 30–70%. Thus, for the main wall, the weight can be calculated as 30% × 2·7 t/m^3 (blocks) and 70% × 1·7 t/m^3. Adding in the parapet wall at 2·7 t/m^3, the total weight reaches 265 t/m. That is, however, dry weight, so buoyant uplift of 87 t/m needs to be subtracted, giving a net weight to resist sliding of 179 t/m.

Sliding resistance is then computed in Table 4 by applying a friction coefficient of μ = 0·78 (see the paper 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.

For a caisson breakwater, a final stage before computing sliding, or overturning resistance, would be to apply the wave-driven uplift force, further reducing the restoring force and/or moment. Here the base of the structure is however permeable, so uplift pressures will not act together to lift the superstructure. These calculations of overturning and sliding in Table 4 have therefore not used F_u.

Despite this optimistic assumption, both sliding and overturning calculations give FOSs below unity – that is, 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 FOSs for both sliding and overturning (Figure 9). These suggest that Stevenson's breakwater would probably have been stable (FOS > 1) only for waves up to H_s < 4·7 m (overturning) or H_s < 6·3 m (sliding). That the reduction of FOS with reducing wave height is not steeper for overturning may be the influence of buoyancy (fixed for a given water level), whereas wave forces reduce directly with reducing wave height.

3.7 Effect of 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/or overturning for wave heights H_s < 5·5 m, even without including the effect of wave uplift forces. It was seen earlier, however, that the steep bed slopes and rock mound will have shoaled and/or broken incident waves, particularly for wave periods $T_p \geq$ 10 s. Not surprisingly, the PROVERBS analysis by Calabrese to identify the occurrence of breaking suggests that breaking is most severe for section A, $P_b \approx$ 29–33%, while further out at sections B and C, $P_b \approx$ 15–23%.

The method for estimating impulsive loads by Cuomo *et al*. (2010) gives meaningful results only for section A, where $F_{imp}/F_{Goda} \approx$ 1·2–2·0, with greater impacts for longer wave periods, $T_p \geq$ 12 s. Impulsive loads are, however, of short duration. Consulting Figure 4 from the paper by Cuomo *et al*. (2011), impulsive forces of two times quasi-static forces are associated at 99% probability with relative rise times $t_r/T_m \approx$ 0·01, so of order 0·1 s. 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

Table 4. Results of analysis of FOSs

Section	H_s: m	T_p: s	Seabed: mCD	H_{si}: m	F_h: kN/m	F_u: kN/m	B_u: kN/m	Mg-B_u: kN m	FOS	
									Sliding	Overturning
A	10	12	–3·0	7·6	1714	851	850	1753	0·8	0·8
B	10	12	–5·4	9·5	2906	1187	1163	2051	0·6	0·6
C	10	12	–7·0	10·2	3192	1158	1373	2244	0·6	0·5
A	8	10	–3·0	6·6	1427	710	850	1753	1·0	1·0
B	8	10	–5·4	7·9	2100	859	1163	2051	0·8	0·8
C	8	10	–7·0	7·7	2314	840	1373	2244	0·8	0·8

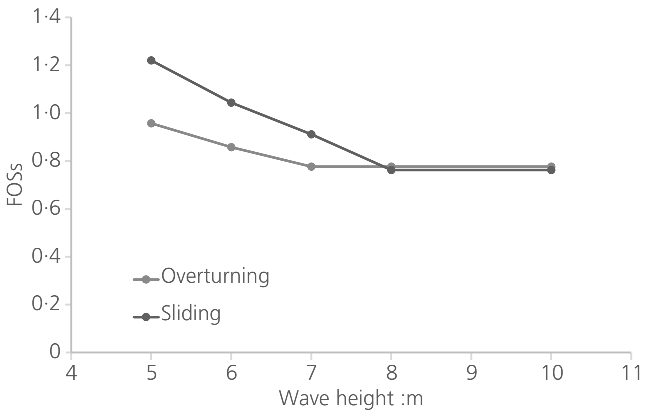

Figure 9. Effect of wave height on FOSs, section B, T_p = 10 s

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.

3.8 Discussion
Stevenson's breakwater at Wick was in a bay exposed to substantial storms from the north, east and south-east. It is probable that even in the absence of validated wave prediction methods, Stevenson might well have expected waves of H_s = 7–9 m. It may have been less clear that waves of $T_p \approx$ 10–14 s 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 would have been greater 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 constraint 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 evidence to allow its analysis, and the global calculations here are enough to demonstrate failure.

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Failure analysis of historic vertical breakwaters, part 2: Alderney, Guernsey and Dover, UK

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Many historic breakwaters failed early in their life, leaving little information by which to analyse or understand their failures. As part of a wider analysis of 'old breakwaters', the first author has analysed the 'stability' of example vertical breakwaters using analytical methods developed over the past 20 years. This analysis is illustrated in this and the companion paper by three case studies: Wick (designed by Thomas Stevenson, failed 1870–1877); Alderney (damaged even during construction, lost its outer length 1865–1889); and Dover (still shows high stability after 110 years). In each of 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 (Probabilistic Design Tools for Vertical Breakwaters) project have been used to explore incidence of wave impact loads, the main momentum loads and impulsive loads. Factors of safety against sliding and/or overturning have been determined for each example over a range of representative wave conditions.

Notation

B^*	dimensionless berm width, B_{eq}/L_p
B_{eq}	effective width of the mound/berm in front of the wall, taken at one-half mound height (m)
C^*	relative reflection coefficient
C_r	reflection coefficient
F_h	horizontal wave force per unit length (kN/m)
H_s	significant wave height (m)
H_s^*	relative wave height, H_{si}/h_s
H_{sb}	significant wave height, broken (m)
H_{si}	incident significant wave height (having taken account of all wave transformations) (m)
H_{ss}	significant wave height, shoaled (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
K_r	coefficient of refraction
K_s	coefficient of shoaling
L	wavelength (m)
L_p	peak period wavelength (m)
m	bed slope (1:x)
P_{av}	average wave pressure (kN/m^2)
P_b	proportion (or percentage (%)) of waves breaking
T	wave period (s)
T_m	mean wave period (s)
T_p	peak wave period (s)
α	front face slope (of foundation mound) (1:x)
β	angle of wave obliquity (°)
γ	wave breaker ratio, H_{sb}/h_s

Note 1: Most definitions are given in full in *The Rock Manual* (Ciria *et al*., 2007), ISO/CD 21650 (ISO, 2007) or the PROVERBS (*Probabilistic Design Tools for Vertical Breakwaters*) book by Oumeraci *et al*. (2001).

Note 2: Some subscripts are condition sensitive, so subscript 'b' indicates breaking but only when dealing with wave conditions – for example, as in H_{sb}; otherwise, 'b' may denote a berm or mound.

Note 3: Proportion or % is defined at the point of use.

1. Introduction

Trade and defence of the UK have depended critically on its harbours. Indeed, trade of coal and foodstuffs around coastal regions was nigh impossible without maritime transport. On exposed coastlines, harbours for supporting this trade were formed by man-made breakwaters. In the absence of robust design methods, many historic breakwaters failed early in their life, leaving few data to analyse or understand their failures. As part of a wider analysis of 'old breakwaters' (Allsop, 2009, 2020; Allsop, *Old British Breakwaters – How Has History Influenced Their Survival?*, PhD thesis in preparation, University of Edinburgh), this and the companion paper (Allsop and Bruce, 2020) describe the stability of example breakwaters using analytical methods developed over the past 20 years. Part 1 summarises the analysis methods and illustrates their application to the breakwater failure at Wick. This paper (part 2) illustrates two further case studies, so together covering

- Wick (designed by Thomas Stevenson, failed 1870–1877)

- Alderney (damaged even during construction, lost its outer length 1865–1889)
- Dover (still shows high stability after 110 years).

For each example, representative cross-sections and approach bathymetry have been derived. Representative wave conditions are transformed to the breakwater toes, including depth-limiting and impulsive breaking effects. Empirical formulae developed during and since the PROVERBS (Probabilistic Design Tools for Vertical Breakwaters) project (Allsop, 2000; Allsop *et al*., 1996; Oumeraci *et al*., 2001) have been used to identify the occurrence of wave impact loads, the main momentum loads and impulsive loads. Factors of safety (FOSs) against sliding and/or overturning have been determined for each example and then compared with reality.

2. Alderney

2.1 History

The Admiralty harbour on Alderney was one of two constructed in the Channel Islands as harbours of refuge; see the paper by Allsop (2020). Alderney Island lies west of the major French naval harbour of Cherbourg in high-velocity tidal streams where flows 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 waves.

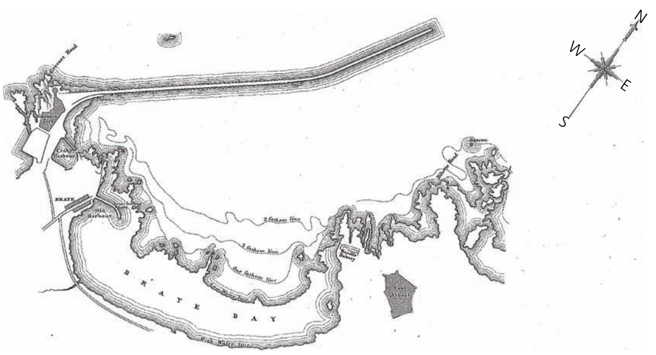


Figure 1. Braye Bay (source: Vernon-Harcourt (1874)). 1 fathom = 1·83 m

Construction of the Admiralty breakwater in Braye Bay (Figure 1) started in 1847, to a design by James Walker, the second Institution of Civil Engineers (ICE) president. The design included a mound to low water, surmounted by blockwork walls with rubble infill (Figure 2). Stone for the mound and walls was quarried from the Mannez quarry on the opposite side of Alderney.

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 125 m steepening the wall, the masonry was set in Medina cement (precursor to Portland cement, made from 1840 at Medina, Isle of Wight) and the foundation for the seaward face of the wall started lower. Having used end tipping hitherto, the new lower mound level required the use of hopper barges, which in turn required shelter from a small construction harbour and a means of loading the barges. In the new works, the rubble mound was not disturbed below −3·7 metres above low water (mLW) (in the absence of the reflecting superstructure). Work to the revised design proceeded 'as soon as diving apparatus and the hopper barges (and method to fill them) were procured' (Vernon-Harcourt, 1874: p. 63).

In 1852 (5 years after construction had started) following repeated breaches and cost increases, Sir Edward Belcher (Redman *et al*., 1874: p. 97) (in the discussion on the paper by Vernon-Harcourt (1874)) notes that Sir Francis Baring summoned him (Belcher) to the Admiralty to tell him '… 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' (Redman *et al*., 1874: p. 97). James Walker was also instructed to go '… in order that he might be there in a gale' (Redman *et al*., 1874: p. 97). Belcher concluded his discussion to Vernon-Harcourt (1874) with the following barbed comment: 'The present works were certainly a credit to British engineers and showed

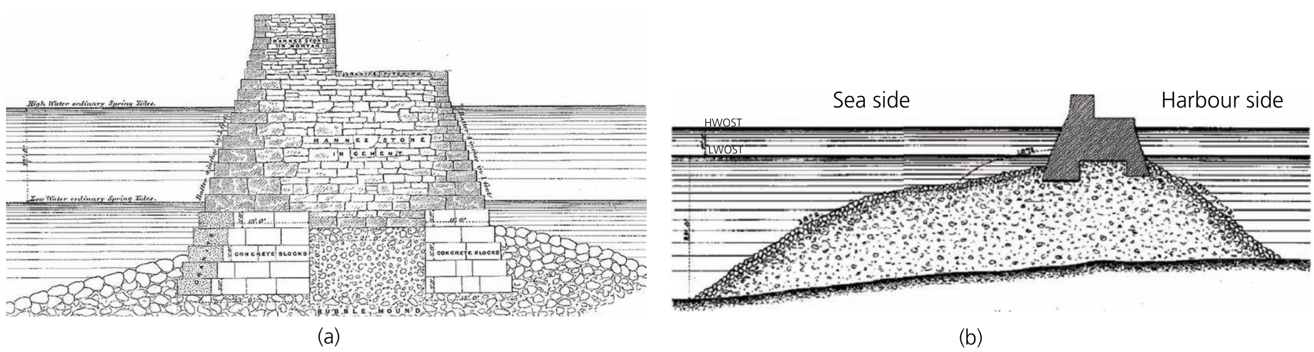


Figure 2. Alderney – section of mound and breakwater wall. Note extended mound from stone dumped down the seaward face of the wall (source: Vernon Harcourt (1874)). 1′ = 1 foot = 0·305 m; 1″ = 1 inch = 25·4 mm

what Englishmen could do when they were determined – whether right or wrong' (Redman *et al*., 1874: pp. 97–98).

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

Following (nominal) completion during 1864 to the full 1430 m, repeated storms in 1865–1869 caused at least nine breaches through the superstructure. Despite repairs, by early 1870, there remained seven locations of damage. Vernon-Harcourt (1874: p. 69) reported that Sir John Hawkshaw (president of the ICE) and Colonel Sir Andrew Clarke were requested by Board of Trade (who had inherited the harbour from the Admiralty) 'to visit Alderney and to report on the best measures for securing permanently', either the whole (1430 m) or an inner (870 m) portion. In 1870, Hawkshaw and Clarke noted the 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 t of stone was 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 seawards of the 870 m division. From 1873, repair and maintenance work covered only the inner length of 870 m, and Partridge (2018: p. 2) reports 'destruction of the seaward end' by 1879 and 'outer section collapsed and submerged' by 1889, leaving a submerged mound at about −4 mLW.

For the shortened section, approximately 20 000 t of stone was dumped annually, only formally ceasing in 1964, although it was interrupted during the German occupation (1940–1945).

In 1987, responsibility for the maintenance of Alderney Breakwater was transferred to the States of Guernsey. Up to 1990, a team of eight men repointed the face of the wall above the mid-tide level, filled cracks and replaced damaged masonry each summer. A team of six engineering divers carried out repair work at the toe, both below and above water. Maintenance costs to 1990 were estimated at around £500 000 per annum. 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 1 m. 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. For the remaining (871 m) section, work was still required to repair breaches in the superstructure.

During winter 1989/1990, storms battered the breakwater for 6 weeks. At its peak on 25/26 January 1990, the storm was about 1:25 year return, with offshore conditions of H_s = 10·0–10·5 m. During the next 6 d, the storm subsided slowly and then rose again to H_s > 7 m. On 11 and 12 February, storm conditions again exceeded H_s = 9 m. 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 d. The repair cost was estimated in 1990 at £1·1 million. Coode & Partners and HR Wallingford explored various potential solutions reported by Allsop *et al*. (1991). Alternative approaches to protecting this breakwater were described by Sayers *et al*. (1998) and recently by Jensen *et al*. (2017).

Figure 3. Admiralty breakwater in 2014 (courtesy of States of Guernsey)

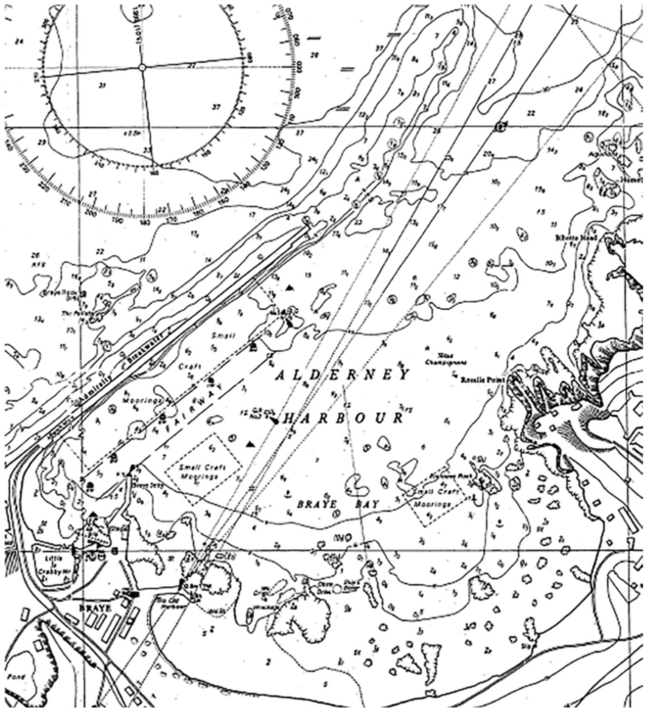


Figure 4. Admiralty breakwater 1974 (courtesy of States of Guernsey)

Table 1. Sections across mound and bathymetry

Section number	Chainage from breakwater root at Fort Grosnez: m
A	130
B	310
C	620
Allsop and Shih (1990)	670
D (breakwater head)	870

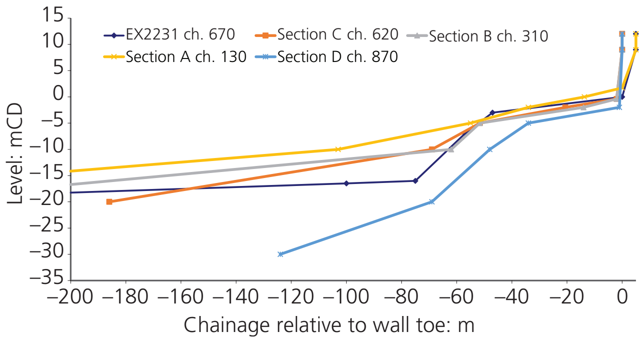


Figure 5. Sections through Alderney Breakwater. The section labelled EX2231 was derived by Allsop and Shih (1990)

The general layout of Braye Bay and detailed bathymetry are shown in Figures 3 and 4. For the FOS analysis here, four cross-sections have been defined running across the seabed and mound orthogonally to the wall. An additional section at chainage 670 m is taken from the report by Allsop and Shih (1990), all listed in Table 1. Depths and offsets from the wall toe are shown in Figure 5 for each section.

2.2 Water levels and wave conditions

The extreme tidal range at Alderney is ~6·8 m, from CD upwards. The main tidal levels listed on Admiralty chart 2845 are

- mean high water springs: +6·3 metres above Chart Datum (mCD)
- mean high water neaps: +4·7 mCD
- mean low water neaps: +2·6 mCD
- mean low water springs: +0·8 mCD.

In their physical model study, Allsop and Shih (1990) used three different water levels: 6·10, 3·15 and 0·90 mCD.

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, being lower and reversing at mid-tide. Wave heights at the breakwater are generally greatest at mid-tide, hence testing by Allsop and Shih (1990) at +3·15 mCD rather than at a higher water level. Wave conditions used for the testing at Wallingford in Table 2 were defined seawards of the approach bathymetry and mound. Wave periods are given in only EX2231 as a mean period, T_m, so an additional column of peak periods, T_p, is given using a nominal conversion, $T_p/T_m \approx 1{\cdot}1$.

Table 2. Wave conditions at Alderney Breakwater (from Allsop and Shih (1990))

Return period: years	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

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, its intersection with the mound. Another 'depth' might be taken at the toe of the breakwater mound. The difficulty here is that the mound was 'fed' with additional rock over approximately 90 years, so its toe has extended well beyond its original position, although it may also have eroded backwards over the past 50 or so years. Inshore of Braye Rocks, an arbitrary depth of −10 mCD may be useful, although this might be extended out to −20 or −30 mCD along the outer part. In the sections in Figure 5, the breakwater 'mound' may be taken some 70 m out from the wall to bed levels of around −10 mCD down to −25 mCD. Approach slopes are generally steep, say, 1:10, particularly at about 50 m out from the breakwater toe.

2.3 Nearshore wave conditions

The bed levels extracted earlier define bed slopes and levels 50 m out at the 'Goda breaking position' ($5 \times H_{so}$ seawards of the breakwater). The 'design' wave heights for this stage of analysis have been taken as the 1:20 year, 1:50 year and extreme conditions in the 1990 studies, as in Table 2.

The waves in Table 3 have been shoaled from their offshore wave height using classical shoaling equations (checked against the graphical method in the *Shore Protection Manual* (USACE, 1977)) and then checked for depth-limited breaking using equations summarised in Figure 2.1 of the report of Owen (1980) as discussed in Part 1 by Allsop and Bruce (2020). These calculations have been run for water levels of +6·1 m and +3·5 mCD, but only the latter are shown in Table 3, 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.

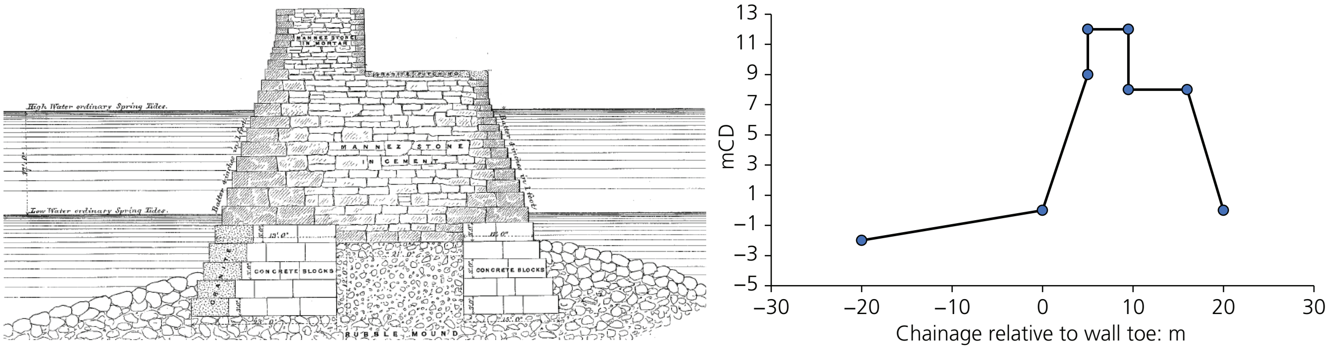
2.4 Breakwater section

The Alderney 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 (Figure 6) is sufficient for the analysis here. The section is simplified to two parts: the parapet wall of 4 m × 4 m and the main wall body at 16 m × 8 m. The wall/mound interface is taken at 0 mCD, and the

Table 3. Results of wave transformation calculations

Return period: years	Section	SWL: mCD	H_s: m	T_p: s	Bed: mCD	Depth: m	K_s	H_{ss}: m	H_{sb}: m	H_{si}: 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
Extreme	A	3·5	9·2	13·2	−8	11·5	1·02	9·4	10·5	9·4
Extreme	B/C	3·5	9·2	13·2	−12	15·5	0·97	9·0	12·0	9·2
Extreme	EX2231	3·5	9·2	13·2	−18	21·5	0·94	8·6	13·8	9·2
Extreme	D	3·5	9·2	13·2	−25	28·5	0·92	8·4	17·1	9·2

As the ‘input’ waves have been derived by numerical modelling, thus deemed to include shoaling, the results of $K_s < 1$ have not been applied in the final calculation of H_{si} in the last column
SWL, static water level
Data in EX2231 from Allsop and Shih (1990)



(a)

(b)

Figure 6. (a) the wall section (after Vernon-Harcourt (1874)); (b) simplified schematisation in FOS analysis

wall crest at +12 mCD. Taking blockwork at 5% porosity and stone of 2700 kg/m^3 gives a net density of 2565 kg/m^3. This simplified section therefore weighs 370 t/m. (A minor correction of adding the density of water for that 5% might be made, but the change will not be significant in this analysis.)

2.5 Wave loads

For initial calculations on the Alderney Breakwater starting with section C (ch. 620), the local water depth in front of the mound (at +3·5 mCD water level) reaches $h_s = 12{\cdot}0$ m + 3·5 m = 15·5 m, and depth over the foundation mound, $d = h_s - h_c = 3{\cdot}9$ m. Thus, $h_c^* = 0{\cdot}75$, a ‘high mound’ in the PROVERBS classification.

The approach bed slopes are steep, although the toe depth is relatively large. Wave heights are, however, relatively large at $H_s^* = 0{\cdot}4$–$0{\cdot}8$ for 1:50 years at all sections (except D, where $H_s^* = 0{\cdot}3$), giving ‘large waves’. At $B_{eq} \approx 0{\cdot}3$–$0{\cdot}5$, the foundation mound is classed as ‘moderate to wide mound’. These calculations suggest that most severe breaking occurs over the mid-length, being broken at section A (ch. 130) by the shallow depths near the root. Contrarily, depths at the head (section D, ch. 870) are greater, and the mound is relatively small, so impulsive breaking over the mound is less.

In these calculations, the foreshortened profile from the report by Allsop and Shih (1990) artificially increased H_{bc}, reducing values of P_i and P_b. These distorted values are therefore not shown in Table 4 or 5. Changing the water level can have a significant effect on the results, so results are shown here for water levels of +3·5 mCD (Table 4) and +6·1 mCD (Table 5).

The total horizontal force, F_h, is calculated by integrating pressures p_1, p_2 and p_3 over the height h_f of the front face. Similarly, the uplift force is calculated by integrating from $p = p_u$ at the front edge to $p = 0$ at the rearward edge, giving $F_u = 0{\cdot}5 p_u B_c$. All force (and pressures) are 1/250 values, $F_{1/250}$, here equal to the average of the top 4 in 1000 events.

2.6 FOS analysis

The simplest stability analysis compares horizontal loads (and uplift) against sliding resistance given by weight and friction. This

Table 4. Wave breaking at SWL = +3·5 mCD

Return period: years	Section	H_s: m	T_p: s	Bed: mCD	Depth: m	H_{bc}: m	H_{si}/H_{bc}	P_i: %	P_b: %
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
Extreme	A	9·2	13·2	–8	11·5	14·1	0·69	1·4	0·5
Extreme	B/C	9·2	13·2	–12	15·5	6·3	1·45	38·8	15·6
Extreme	D	9·2	13·2	–25	28·5	6·6	1·39	35·6	14·2

Table 5. Wave breaking at SWL = +6·1 mCD

Return period: years	Section	H_s: m	T_p: s	Bed: mCD	Depth: m	H_{bc}: m	H_{si}/H_{bc}	P_i: %	P_b: %
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
Extreme	A	9·2	13·2	–8	14·1	4·6	2·04	61·9	25·4
Extreme	B/C	9·2	13·2	–12	18·1	4·2	2·19	65·8	27·1
Extreme	D	9·2	13·2	–25	31·1	7·9	1·16	22·7	8·8

requires some simplifying assumptions but is 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, initially as a whole. For Alderney, the section (Figure 6) can be divided into three parts.

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

Most of the outer section is formed by close-fitting blocks, so the weight of the main wall and parapet is 144 m^2/m × 2·565 t/m^3 = 370 t/m. Buoyant uplift will act to either 6·1 or 3·5 mCD.

Converting weight and buoyant uplift to kilonewtons per metre, net weight forces of 2700 and 3060 kN/m are obtained at water levels of 6·1 or 3·5 mCD.

For sections B and C (chainages 310 and 620 m), assuming a friction coefficient μ = 0·78 (see the paper by Hutchinson *et al.* (2010)) gives net sliding resistances of 2110 or 2390 kN/m to set against the wave loads in Tables 6 and 7, respectively. The resulting FOSs, without and with the effect of wave uplift, are summarised in Tables 8 and 9.

Table 6. Wave loads by Ito (1971) and Goda (1985, 2000) at SWL = +3·5 mCD

Return period: years	Section	p_{av}: kN/m^2	$F_{h(Ito)}$: kN/m	η^*: m	p_1: kN/m^2	p_2: kN/m^2	p_3: kN/m^2	$F_{h(Goda)}$: kN/m	$F_{u(Goda)}$: kN/m
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
Extreme	A	169	3374	26·2	199	192	198	2380	880
Extreme	B/C	120	2876	24·8	192	181	189	2290	830
Extreme	D	76	2823	24·8	165	135	159	1945	830

Table 7. Wave loads by Ito (1971) and Goda (1985, 2000) at SWL = +6·1 mCD

Return period: years	Section	p_{av}: kN/m^2	$F_{h(Ito)}$: kN/m	η*: m	p_1: kN/m^2	p_2: kN/m^2	p_3: kN/m^2	$F_{h(Goda)}$: kN/m	$F_{u(Goda)}$: kN/m
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
Extreme	A	134	2689	25·4	201	191	198	2400	850
Extreme	B/C	106	2547	24·8	181	167	176	2140	830
Extreme	D	72	2662	24·8	161	127	152	1880	830

Table 8. FOS (sliding) for sections B/C, no wave uplift

Return period: years; section	SWL: mCD	Net sliding resistance: kN/m	Wave force, F_h: kN/m	FOS
20; B/C	3·5	2280	1760	1·29
20; B/C	6·1	1870	1670	1·12
50; B/C	3·5	2280	2060	1·10
50; B/C	6·1	1870	1950	0·96

Table 9. FOS (sliding) for sections B/C, with wave uplift

Return period: years; section	SWL: mCD	Net sliding resistance: kN/m	Wave force, F_h: kN/m	FOS
20; B/C	3·5	1755	1760	1·00
20; B/C	6·1	1350	1670	0·81
50; B/C	3·5	1680	2060	0·82
50; B/C	6·1	1275	1950	0·65

Taking the more optimistic scenario, without wave-driven uplift forces and assuming the breakwater wall to act monolithically, then it should slide under a 1:50 year wave condition at a high water level of +6·1 mCD but should just resist sliding at the lower water level of +3·5 mCD (Table 8). At section A, the FOSs calculated here are lower, but those simple calculations have not included the weight of additional material behind the simple section considered here – that is, at the breakwater root work area and the inclined slipway – all of which will increase sliding resistance.

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

The 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

Table 10. FOS (overturning) for sections B/C, no wave uplift

Return period: years; section	SWL: mCD	Net overturning resistance: kN m/m	Overturning moment, M_{total}: kN m/m	FOS
20; B/C	3·5	39 500	7920	5·0
20; B/C	6·1	39 500	13 100	3·0
50; B/C	3·5	39 500	8080	4·9
50; B/C	6·1	39 500	13 300	3·0

Table 11. FOS (overturning) for sections B/C, with wave uplift

Return period: years; section	SWL: mCD	Net overturning resistance: kN m/m	Overturning moment, M_{total}: kN m/m	FOS
20; B/C	3·5	39 500	17 000	2·3
20; B/C	6·1	39 500	22 000	1·8
50; B/C	3·5	39 500	18 200	2·2
50; B/C	6·1	39 500	23 400	1·7

Table 12. Impulsive and quasi-static force maxima, methods of Cuomo *et al*. (2010, 2011)

Return period: years; section	SWL: mCD	Impulsive force, F_{imp}: kN/m	Quasi-static force, F_{qs}: kN/m	F_{imp}/F_{qs}
20; A	6·1	8670	2720	3·2
20; B/C	6·1	3170	2640	1·2
20; D	6·1	730	2640	—
50; A	6·1	14 420	3650	3·9
50; B/C	6·1	5990	3410	1·8
50 D	6·1	7850	3410	—
Extreme; A	6·1	17 220	4260	4·0
Extreme; B/C	6·1	8160	4080	2·0
Extreme; D	6·1	10 340	4080	1·2
20; A	3·5	33 850	2940	11
20; B/C	3·5	10 800	2640	4·1
20; D	3·5	6300	2640	2·4
50; A	3·5	50 500	3910	13
50; B/C	3·5	16 500	3410	4·9
50; D	3·5	10 700	3410	3·1
Extreme; A	3·5	57 800	4540	13
Extreme; B/C	3·5	20 200	4080	4·9
Extreme D	3·5	13 600	4080	3·3

wave uplift forces. The coefficient of friction μ = 0·78 was derived by Hutchinson *et al*. (2010) from full-scale pull tests on a roughened caisson base on rock.

The overturning results in Tables 10 and 11 are substantially more reassuring with the FOS always above 1·7 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.

2.7 Impulsive loads

Impulsive loads have been calculated in Table 12 using the methods by Cuomo *et al*. (2010, 2011) (see Figure 4 in part 1 (Allsop and Bruce, 2020)), from which it can be seen that impulsive loads greater than (say) two times the Goda load will act only for a duration of ≈$0{\cdot}01T_m$, so on the order of 0·1 s. 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.

A further aspect of impulsive loads is that they are of limited spatial extent, so even if effective in causing (very short duration) excess loadings, these will be confined to limited areas. That said, the low FOSs, even in Tables 8 and 9, do provide justification for the historical failures, and the high predictions of impulsive loads in Table 12 indicate the significant loadings to which this breakwater is subject.

3. Dover

3.1 History of the Dover harbour breakwaters

A National Harbour had been mooted at Dover for many years. Wilson (1920) notes that the 1840 Royal Commission favoured a deep-water harbour in Dover Bay of 450 acres (18·2 km^2), estimated cost of £2 000 000. The 1844 Royal Commission reconsidered whether to establish a harbour of refuge here, requiring any site to

- deliver ease of access for vessels 'requiring shelter from stress of weather'
- deliver armed vessels in event of hostilities, both offensive and defensive
- 'possess facilities for ensuring its defence' against attack.

While the harbour at Dover was in theory to be 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 (21 km^2) out to 7 fathoms (12·8 m). The outer breakwater was to be aligned with the tidal currents (to reduce siltation).

The commissioners reported in 1846 in favour of Rendel's design with a vertical wall. Vernon-Harcourt (1885) noted not only damage to sloping breakwaters at Cherbourg and Plymouth but also the shortage of experience in concrete. However, given the chalk bottom, the absence of local rock 'and a moderate depth, the upright wall was the best system to adopt' (Vernon-Harcourt, 1885: p. 332). A series of contracts in 1847, 1854 and 1857 extended Admiralty Pier to 640 m by 1871.

Admiralty Pier was formed by 7–8 t concrete blocks with stone facings on the outer faces. The main wall was 'surmounted by a high parapet, overhanging considerably to the seaward' (Wilson, 1920: p. 33). Wilson shows a section of this parapet wall of a single column of blocks, about 1·5 m thick, with a relatively slight recurve to modern eyes. In January 1877, about 300 m of this parapet was swept away. Wilson blames the curved overhang, although the slender nature of the up-stand wall and the absence

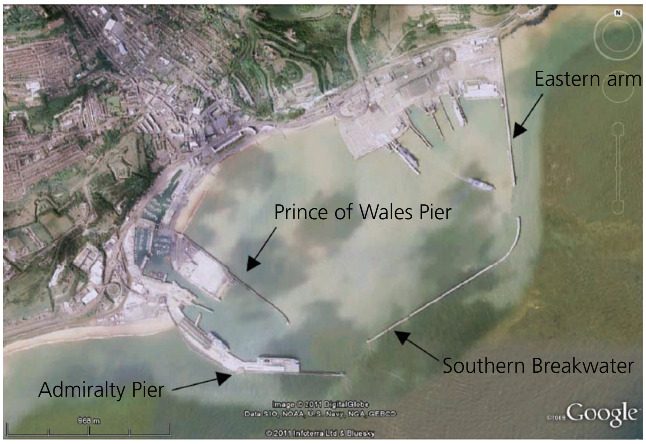


Figure 7. Dover Harbour, present day (courtesy of HR Wallingford and Google Maps (©2009 Google; ©2011 Digital Globe, Data SIO, NOAA, US Navy, NGA, GEBCO; ©2011 Infoterra Ltd and Bluesky))

of tensile reinforcement against bending must surely have contributed. The damaged section was rebuilt with a significantly thicker (about 3·3 m) 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 7) by

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

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 reduce siltation. The Coode design was rapidly approved by the Admiralty, and a construction contract was let to S Pearson & Son in November 1897.

The new walls (Figure 8) were formed by 24–40 t concrete blocks (2·3 m wide and 1·8 m high, depth from 2·4 to 4·0 m) 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. 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.

For the foundation layers, underwater blocks were set by divers, placed tightly without mortar. Above the low water course (a band 1·8 m high centred on low water ordinary spring tides (LWOST)), the next four courses were grouted by 2:1 Portland cement 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.

The Eastern Breakwater, termed the East Arm, projects south for 900 m in section similar to the Admiralty Pier extension, although the parapet wall was lower at +8·8 m LWOST. Foundation blocks for the East Arm wall were laid direct on the chalk inshore, or the chalk marl/flint matrix further seawards, down to −16·2 m 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 1284 m 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

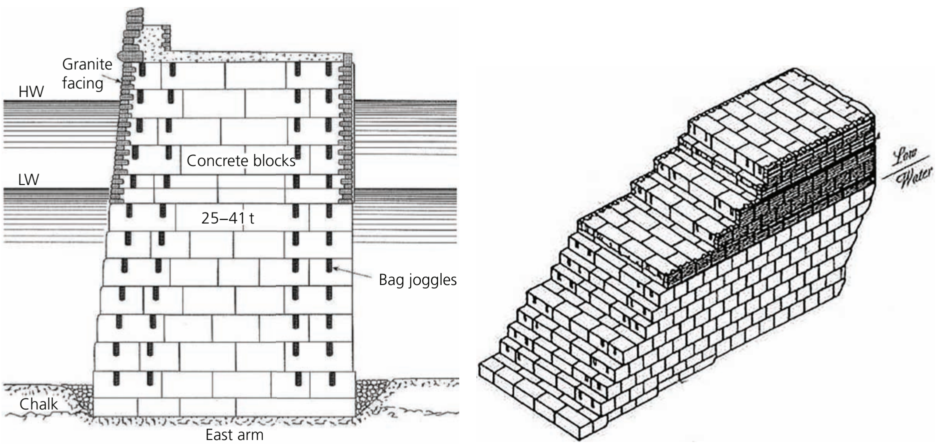


Figure 8. (a) Schematic section of Dover Southern Breakwater; (b) Arrangement of blockwork, after Cruickshank *et al.* (2011). HW, high water; LW, low water

length connected the eastern end to the main run using curved (radial) blocks to preserve block tightness.

Ahead of block placing (see Figure 9), the seabed was prepared by excavating 1·5 m of surface material. The chalk or chalk/flint matrix was loosened by a cast-iron breaker dropped from the leading (60 t) Goliath. The final 0·3 m was removed by four men within a 35 t diving bell, excavating a strip about 4·6 m wide for two rows of blocks. The bell passed over each strip twice to give a coarse levelling, 'within a few inches', and 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. Diver working was limited to about 4–5 h each tide, during which six blocks were placed per hour at best. Blocks above were set by masons during the 2–3 h 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/rope, pointed in neat (quick-setting) cement, to avoid loss of jointing/bedding mortar downwards.

3.2 FOS analysis

In the course of a residual life assessment study for the Dover Harbour Board, Cruickshank *et al*. (2011) analysed wave loads on different parts of the main breakwaters at Dover (load and stability calculations by the first author). Selected calculations are repeated and extended here.

Figure 9. Goliath crane on staging (courtesy of Dover Harbour Board)

Table 13. Wave conditions (Allsop and Shih, 1990)

Return period: years	Wave height, H_s: m	Wave period, T_m: s	Direction: ° north
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[a]	6·2	10·0	—
500[a]	6·5	10·3	—

[a] Results extracted from original calculations but not shown in that report

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 7·4 up to 9·6 mCD (0 m Ordnance Datum Newlyn = 3·67 mCD). Wave conditions were extracted from previous wave modelling to give predicted wave heights (H_s), period (T_m) and direction (° north) for return periods from 0·1 to 100 years. 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 13.

Moderate simplifications were made to the example section for this analysis. A seabed level at −11 mCD was chosen, with a wall crest at +15 mCD. The wall was taken as vertical on both faces and of width 15 m – the slight batter will alter neither the loading materially nor the sliding resistance. The analysis by Cruickshank *et al*. (2011) used a precautionarily light density of ρ_c = 2·14 t/m^3 for the assemblage of concrete blocks. A range of water levels was explored up to +9 mCD.

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

For the 1:200 and 1:500 year returns, horizontal loads alter very slightly with water level (Figure 10), reaching a maximum at a frequent water level of +6 mCD.

Of most interest are calculations of FOS against sliding (Figure 11) using μ = 0·8, realistic where joggle bags cross layers, or any settlement led to any steps between layers. In all instances analysed, the FOS remains well above unity. For μ = 0·8, FOS > 1·5 for all cases modelled. This analysis is surprisingly reassuring, given that even the wave load analysis by Sainflou (1928) was available only in 1928 and Goda's first version was

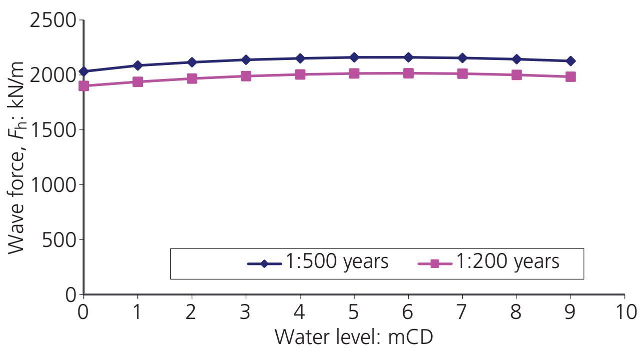


Figure 10. Total horizontal wave force, F_h

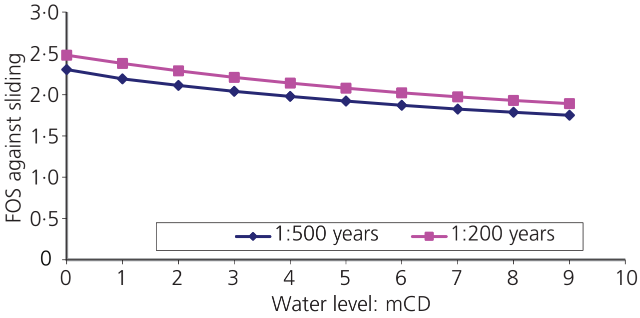


Figure 11. FOSs against sliding, $\mu = 0{\cdot}8$

first published in English only in 1974 (Goda, 1974). The original designers probably therefore had no quantitative load prediction method other than their previous experience!

4. Comments on the stability of these breakwaters

The early failure of the end of Wick Breakwater (discussed in part 1 of this paper) 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, having delayed breaking. Wave loads onto the wall were 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 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 the 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 past 25 years to breakwaters built between 1850 and 1910 to analyse the stability of three structures. This has required some simplifications, but even so, the FOS analysis has successfully matched their performance.

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