STABILITY ANALYSIS OF OLD BREAKWATERS: CASE STUDIES OF FAILURES AND SUCCESSES

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INTRODUCTION

Until about 1930, analysis of wave loads on vertical breakwaters was based on trial and error. Russell[1] noted it was unfortunate that "*the young engineer …should be left to be guided entirely by circumstances, without the aid of any one general principle*." Stevenson[2] noted "*the engineer has always a difficulty in estimating the force of the waves with which he has to contend*." Wave force formulae by Sainflou[3] and Goda[4] improved design methods, but were 50 to 100 years too late for many 'old' breakwaters, and do not apply to many composite breakwaters – ignoring the seminal influence of shoals or mounds on wave breaking and impulsive loadings.

This paper presents case studies using empirical methods developed over the last 20-30 years to calculate loads and stability of example 'old' breakwaters including: Wick (failed before completion); Alderney (multiple breaches during construction, and only survives to ½ of its original length); and Dover (survives with substantial Factors of Safety). The screening analysis[5] used empirical methods developed over the previous 20 years[10], summarised in the case studies[6,7]. Methods used here are empirical (no numerical modelling) so that the calculations can easily be repeated by local engineers.

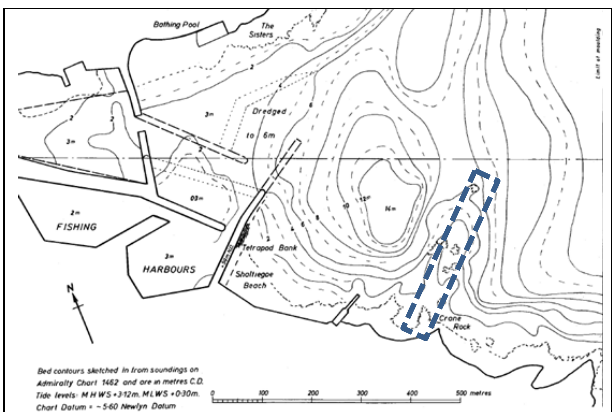

Figure 1 Wick Bay with location of Stephenson's breakwater (*after Hydraulics Research Station EX706*)

WICK

Wick in north-east Scotland was a major fishing harbour in the 1700s and 1800s. Telford expanded the harbour in 1811 and further in 1825-1834. A new outer breakwater by D & T Stevenson begun in 1863 was damaged in October 1868, with 75m lost. In February 1870 the outer part was destroyed in a storm described by Paxton[8].

The general bathymetry of Wick Bay (Fig 1) shows the shoal of Crane Rocks along which the Stevenson breakwater was built. The dashed box indicates the Stevenson breakwater and is used to estimate seabed levels for wave transformation calculations. Paxton (2009) suggests that Stevenson "would have expected waves of (H_s=) 7-9m". In 1975, HRS[9] derived 1:1 year H_{max}=12m, (H_s=6.7m) and 1:50 year H_{max}=18m, (H_s=10m). Wave periods used were T=14s down to T=7s for frequent conditions. Stability calculations used H_s=8m and 10m.

To derive incident conditions, waves must be shoaled and/or broken over the last 50-100m. Sections were taken across the line of Stevenson's breakwater normal to the -10mCD contour, chainages of 100m, 180m, and 250m from the shoreline. Bed slopes average 1:10-1:20.

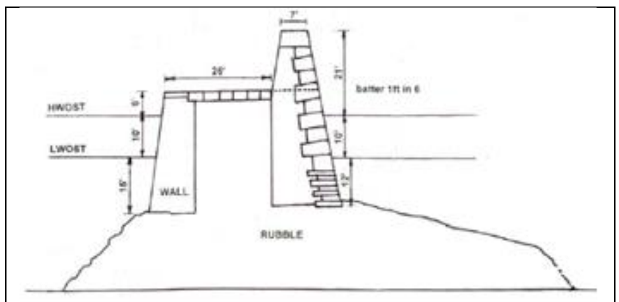

Figure 2 Stevenson's breakwater at Wick (*after Paxton (2009)*

Both sliding and overturning Factors of Safety for the section in Fig 2 fell below unity for all of the 1870 hindcast conditions[6]. Even for smaller waves, this breakwater would only have been stable (FoS>1) for waves H_s<4.7m (overturning) or H_s<6.3m (sliding).

ALDERNEY

The Admiralty breakwater at Alderney was constructed 1847-64 to a design by James Walker, including a mound to low water, surmounted by blockwork walls with rubble infill to a projected total length of 1430m. By 1849, experience over two winters[11] had shown up significant weakness in Walker's design with frequent breaches of the breakwater wall. The section was amended from ch.125m steepening the wall, masonry was set in Medina cement, and the seaward wall foundation set lower.

This construction continued to ch. 823m by 1856. The design was then revised again, further lowering the wall foundation level, now easier with the availability of divers[11]. Following (nominal) completion to the ch.1430m in 1864, repeated storms in 1865 to 1869 caused at least nine breaches through the superstructure. Sir John Hawkshaw and Col. Sir Andrew Clarke were requested *" to report on the best measures for securing permanently*", either the whole (1430m) or an inner (870m) portion. They noted instability of the mound and suggested deposition of additional rubble or concrete blocks. About 300,000 tons of stone were tipped between 1864 and 1871, after which Board of Trade abandoned the outer length. Partridge(12) notes up to 20 breaches or defects by 1873, most seaward of 870m. From 1873, repair and maintenance covered only the inner length of 870m, Partridge(12) reports "*destruction of the seaward end*" by 1879 and "*outer section collapsed and submerged*" by 1889, leaving a submerged mound at about -4mLW.

In 1987 responsibility transferred to States of Guernsey. Each summer a team of 8 repointed the face of the wall above mid-tide level, filling cracks and replacing damaged masonry, whilst 6 engineering divers repaired the wall toe, both below and above water.

During winter 1989/90, storms battered the breakwater for six weeks. At its peak on 25/26 January 1990, offshore waves reached H_s=10 to 10.5m. That storm subsided to H_s >7m but on 11/12 February again exceeded H_s > 9m. This cracked the masonry facing, and a large cavity was formed in the wall which was breached by an explosive failure.

Since that 1990 damage event, routine / recurrent strengthening of the wall appears to have reduced both occurrence and severity of damage.

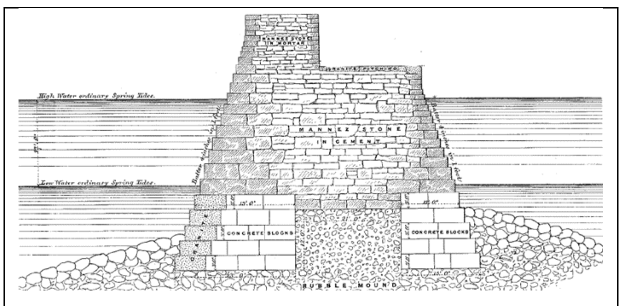

Figure 3 Alderney breakwater wall section used in Factor of Safety analysis *(after Vernon-Harcourt, 1873)*

For calculations starting with ch. 620, the local water depth in front of the mound (at +3.5mCD water level) reaches h_s = 12m+3.5m = 15.5m, and depth over the foundation mound, $d=h_s-h_c$=3.9m. So h_c^* = 0.75, a 'high mound' in the PROVERBS classification(10). Calculations of wave breaking and resultant impulsive wave loads suggest that probability of breaking onto the Alderney wall reaches $P_{b\%}$ = 6-25%. in 1:50 year waves. Factors of Safety (sliding, ignoring dynamic up-lift) for 1:50 year fall between *FoS* = 0.96 and 1.1 depending on water level. Including the effects of wave up-lift forces on the wall section would however drop *FoS* = 0.65 to 0.8, i.e. failure.

DOVER

The single pier at Dover did not give adequate shelter from easterlies, and in 1895, the Admiralty requested Coode, Son & Matthews to prepare surveys and drawings to facilitate expansion by extending

Admiralty Pier by 610m;
adding a detached breakwater of 1284m;
adding an Eastern Arm of 1012m.

The Coode design was rapidly approved, and a contract was let to S Pearson & Son in November 1897(13). The new walls (Fig. 4) were formed by 24-40t concrete blocks (2.3m wide and 1.8m high, depth from 2.4 to 4m) to accommodate the 12:1 batter and ensure adequate bonding. Jointing was strengthened by half-height joggle joints, filled by 4:1 concrete rammed into canvas bags. Around the outer ends, tensile connections were 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 1.8m high centred on LWOS) 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.

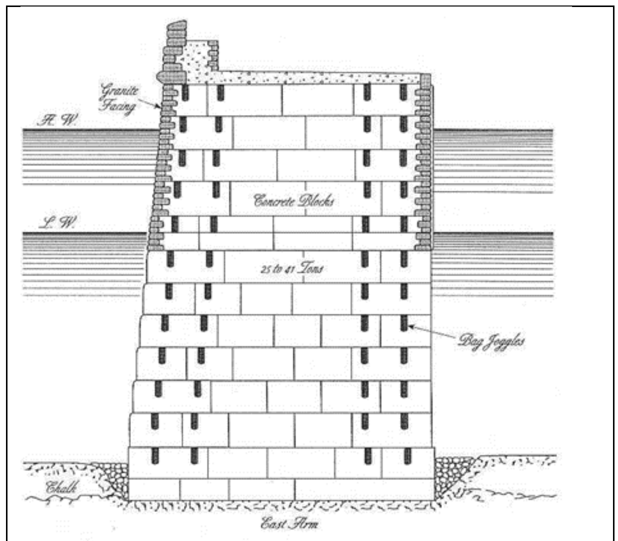

Figure 4 Construction of Dover outer breakwater wall section *(after Vernon-Harcourt, 1873)*

In the course of a residual life assessment study for Dover Harbour Board (DHB), the first author(15) analysed wave loads and stability calculations(7). The range of water levels covered return periods of 1-1000 years at dates of 2000 and 2060, giving water levels of: 7.4mCD up to 9.6mCD (0mODN = 3.67mCD). Wave conditions were extracted from previous wave modelling to give predicted wave height, period, and direction (°N) for return periods from 0.1 year to 100 year. As might be expected, the largest waves are from the south and south-west, and will hit the Admiralty pier extension at normal incidence, β≈0°. For load calculations, moderate simplifications were made to the example section. A bed level at -11mCD was chosen, with a wall crest at +15mCD. The wall was taken as vertical on both faces and of width 15m – the slight batter will neither alter the loading materially nor the stability. A precautionarily light density of ρ_c= 2.14t/m^3 had been taken for the assemblage of blocks. A range of water levels were explored up to +9mCD (+5.33mODN).

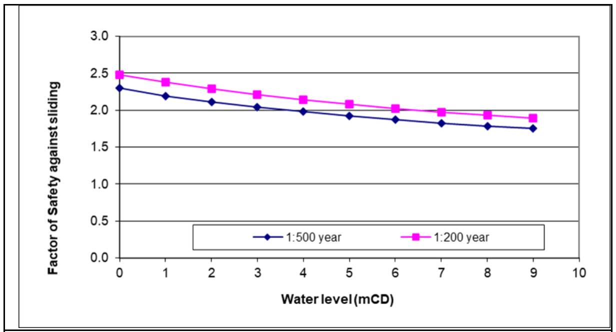

Figure 5 Factors of Safety against sliding, μ = 0.8

Wave loads were calculated[7] using Goda's[16] method, 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. Even for the 1:200 year conditions and elevated water levels, these stability calculations give *FoS* = 1.4-1.8, so fully stable.

CONCLUSION

These calculations support and illustrate conclusions from the historical review in Allsop's thesis[1], mainly focussed on papers and discussions in ICE Proceedings, and the key textbooks by Stephenson[2], Vernon-Harcourt (1885) and Shield (1895). The calculations of wave forces and stability[5,6,7] use empirical methods initiated in the PROVERBS project[10] and refined since[5].

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