Alderney Breakwater, a developed rehabilitation solution

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Abstract
Alderney Breakwater, also known as the Admiralty Breakwater as it was conceived and developed by successive Admiralty Boards, was built in the period from 1850 to 1864. The breakwater originally had a length of 1430 m; although by 1872 continuing damage resulted in unsustainable maintenance costs and as a consequence the Admiralty decided to maintain only the inner 871 m.

The breakwater has been assessed in its existing situation as being unstable for its design conditions and, furthermore, breaches have occurred to the masonry on a number of occasions, for example in 1961 and 1990. A significant volume of work has been carried out to maintain the structure in response, with maintenance works consisting of dumping stones in front of the breakwater and local repair of the masonry structures. The current owner, The States of Guernsey, has spent about £ 20M in repairs over the period from 1984 to 2014.

In 2001/02, The States of Guernsey launched a competition for development of a rehabilitation solution for the breakwater. A Danish consortium consisting of Pihl (contractor) and COWI (consultant) participated and was eventually declared the winner. This developed rehabilitation solution is described in the present paper.

Introduction

Background
Alderney is one of the Channel Islands and it is part of the Bailiwick of Guernsey, a British Crown dependency. The vast increase in maritime trade during the early Victorian period led the Government to propose a breakwater for Alderney harbour. The proposals were developed by successive Admiralty Boards and the Alderney Breakwater was built in the period from 1850 to 1864. The harbour was originally built as a facility for the Royal Navy, although the breakwater has long served as protection for commercial and fishing vessels.

The breakwater originally had a length of 1430 m although by 1872 continuing damage resulted in unsustainable maintenance costs and as a consequence the Admiralty decided to maintain only the inner 871 m. It appears, as shown in the Admiralty chart in Figure 1, that the abandoned outer section of the breakwater is still visible as a submerged ruin of the rubble foundation.

The breakwater structure comprises: (1) a rubble foundation consisting of quarry rocks in a very flat profile, and (2) a superstructure consisting of an outer masonry structure with an access roadway on top. In the core the structure is filled with rubble, see Figure 6. The breakwater has been assessed in its existing situation as being unstable for its design conditions and over the years a significant volume of work has been carried out to maintain the structure. Substantial breaches have occurred to the masonry a number of times, most recently in 1961 and 1990, while many other problems have been experienced over the last 150 years since construction.

Routine maintenance works typically consists of dumping stones in front of the breakwater and local repair of the masonry structures. The current owner, The States of Guernsey, has spent about £ 20M in repairs over the period from 1984 to 2014.

It is thus a general conclusion by professionals that if no major project work is undertaken the mound will continue to deplete. This will eventually lead to new breaches of the superstructure and perhaps further failure during future storm events.

Competition for breakwater rehabilitation (2001)

In 2001/02, The States of Guernsey launched a competition for development of a rehabilitation solution for the breakwater. A Danish consortium consisting of Pihl (contractor) and COWI (consultant) participated and was eventually declared the winner.

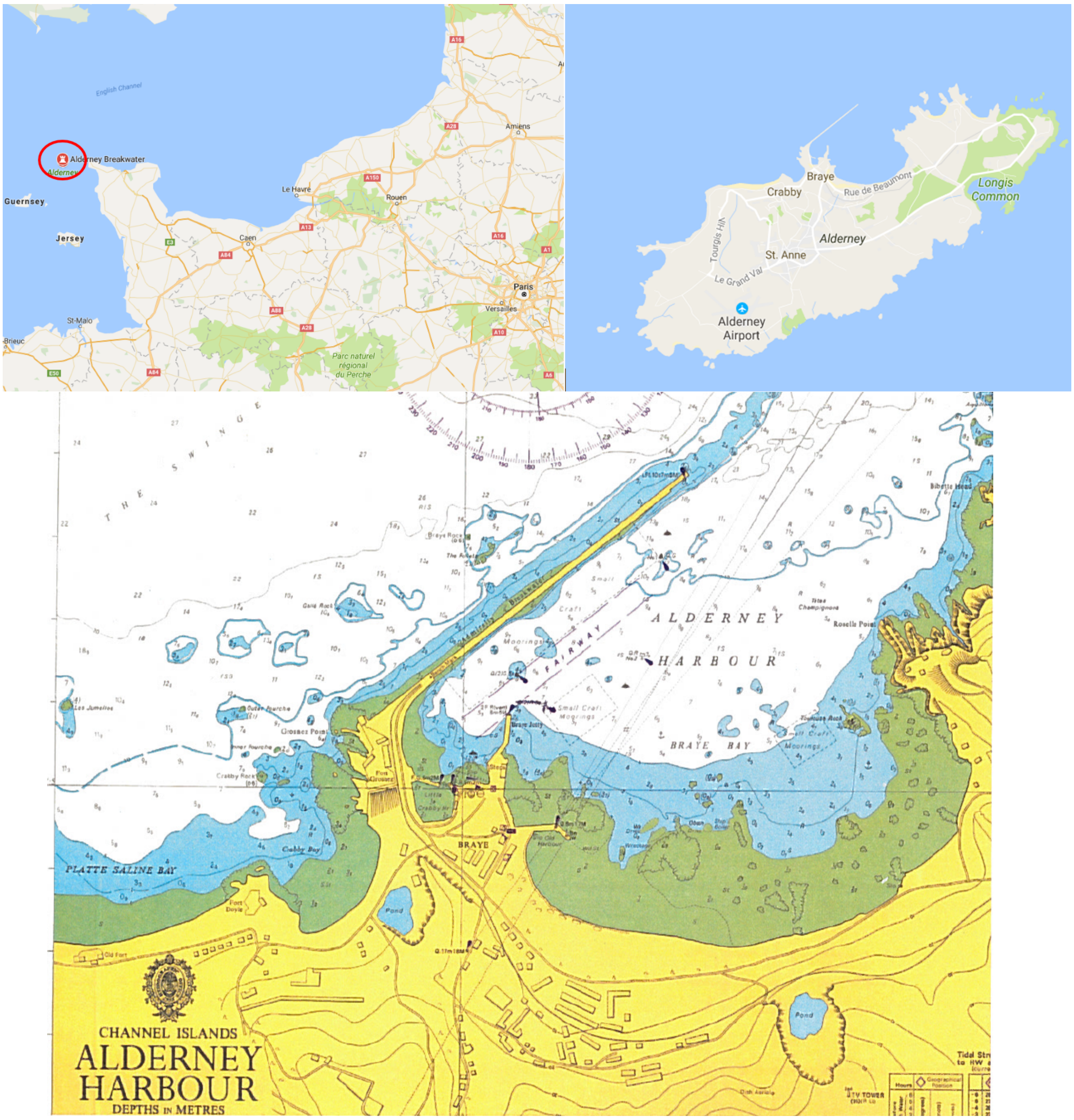

Figure 1 Location of Alderney Breakwater (up) Admiralty Chart of Alderney Harbour and Breakwater (below)

Alderney Breakwater is an internationally renowned historical monument and well known among breakwater and port engineers worldwide. It has previously been described in a number of publications (Allsop NWH et al, 1991 and Sayers, P.B et al, 1998). The developed rehabilitation solution took into consideration a number of requirements: First, the solution should provide long term performance with very limited maintenance. Secondly, due to the nature and heritage value of the breakwater, it was found essential that the solution had the most natural appearance possible.

The photo in Figure 2 gives a good impression of the overwater part of the breakwater and the construction technique using masonry that was common at that time before the introduction of concrete. Figure 3 shows a typical cross section of the existing and the proposed upgraded breakwater. Figure 4 shows the seaward side and how the breakwater is exposed to waves impinging on the front and causing run-up and overtopping even for relatively small waves, as on the day the picture was taken. In severe storms very severe green water overtopping occurs over the entire breakwater length.

Figure 2 Alderney Breakwater roadway and rear-side of wave wall (Courtesy Adrian Findlay – from: http://www.findlays.net/photo/clonque-12-02/index-all.html)

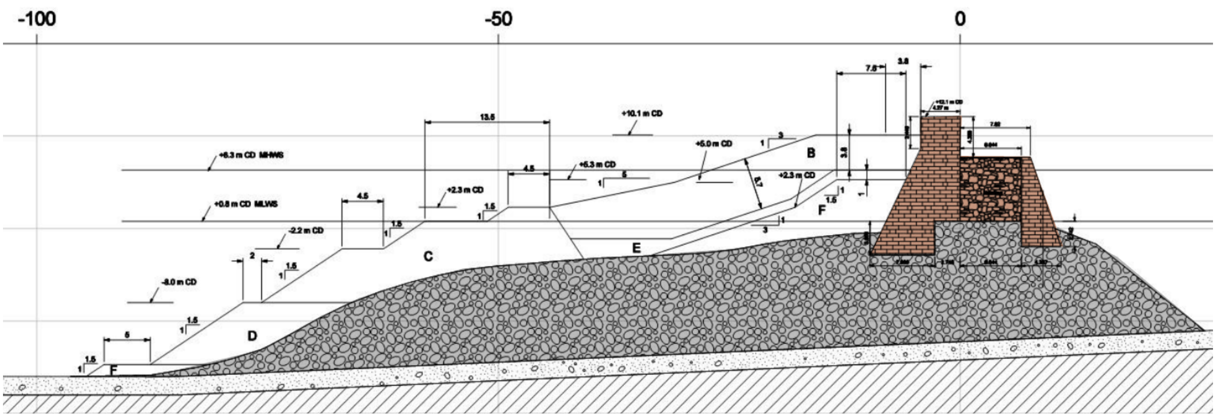

Figure 3 Cross section showing the existing breakwater (grey and brown) and the proposed upgraded breakwater

Figure 4 Front-side of the breakwater (Courtesy Adrian Findlay – from: http://www.findlays.net/photo/clonque-12-02/index-all.html)

Pihl/COWI studied the history of the breakwater and the recent (last 10 to 15 years before 2002) studies and investigations into how to safeguard the breakwater. Following these studies it was decided that the most feasible way to secure the existing breakwater with minimum interference with its present state would be to construct a further large rubble mound in front of the structure.

This conclusion was based upon the following reasoning:

- The breakwater is essential for the future of the harbour and Alderney.
- The adopted solution should be a viable long-term technical solution with limited maintenance requirements.
- It should respect the 150 year old breakwater, a world class historical monument and engineering masterpiece that should be preserved for its heritage value.
- The site was inspected and it is clear that the breakwater rear-side is visible from many locations on Alderney. In order to maintain the present historical esthetical appearance, its rear-side should be kept unchanged if possible.
- The breakwater sea-side is primarily visible from the sea. It was then concluded that the seaward side could be modified if needed to make a viable rehabilitation solution.

Once it was concluded that the optimum solution was to strengthen the sea-side, both rock and concrete solutions were discussed and assessed. For both aesthetic as well as durability reasons quarry rock was selected as the preferred construction material. The rock solution presented in the following has thus been developed to mitigate the following problems:

- As the rocks presently dumped in front of the masonry wall are too small to be stable, the rocks are moving and rolling during severe wave action and the rocks are thus deteriorating and becoming smaller with degradation due to abrasion.
- Severe wave slamming and normal wear and tear over 150 years have resulted in mortar between blocks being removed and failure of masonry joints. As a result the superstructure is pervious and blocks break loose. The solution developed reduces significantly the wave slamming and thereby reduces wear and tear. It has been discussed whether it would be possible to re-inject mortar into the joints, but it is found impractical and not really required if the waves will be absorbed in the rubble mound rather than impinging onto the wall and its weak foundation. Furthermore, the large mound will keep the front masonry wall in place even if the mortar further deteriorates.
- The rubble mound is losing material due to abrasion of stones during wave action and the superstructure has occasionally been undermined. Furthermore, due to the erosion of stones in front of the wall irregular settlements occur that open up joints between blocks. The rock mound solution adopted will remedy this. The existing weak rubble mound will be covered by several metres of rock material of high quality and durability and hence the existing mound will be fixed. The differential settlements should then be reduced; and the presence of the new mound will significantly reduce the wave impacts and wave motion in the mound and in the partly permeable breakwater superstructure.

Design data

The design data in terms of water level and wave data appears in the tables below. The astronomical tide is quite large at Alderney, up to about 6-7 m at maximum spring tide. Due to its location and exposure the design waves are large and the 50 year Return Period waves is about H_s= 7.2 m with a mean period of about 12.3 s, which corresponds to peak periods of about 16 to 17 s.

Table 1: Tidal data

	Tidal level (m chart datum)
Highest Astronomical Tide (HAT)	+7.0
Mean High Water Springs (MHWS)	+6.3
Mean High Water Neap (MHWN)	+4.7
Mean Low Water Neap (MLWN)	+2.6
Mean Low Water Spring (MLWS)	+0.8
Lowest Astronomical Tide (LAT)	+0.2

Table 2: Wave statistics

Return Period (years)	**H_s (m) (from 330°)**	**T_m (m)**
0.1	4.0	9.6
1	5.2	10.7
10	6.2	11.8
50	7.2	12.3
100	7.6	12.8
2000	9.5	14.2

Rehabilitation solution

A number of solutions were considered and among these the solution with a large berm constructed from quarry rock presented in Figure 5 was found to be the most attractive. This cross section is approximately 100 m wide and only slightly wider than the existing breakwater. The water depth in front of the breakwater toe is about 10 to 13 m in the existing situation and is increased for the rehabilitation solution where it extends to 16 m depth in chainage 200 (see profile in Figure 6) and to 29 m at the breakwater roundhead. The front berm on the trunk of the breakwater is composed of various categories of quarry rock with the largest gradation being 12 to 24 t rocks with an average of 18 t. The material is granite with specific gravity of about 2.65 t/m³. These very large rocks are planned to be transported on barges from Sweden, where such rocks are available. Larger rocks 20-30 t, with an average of 24 t, are used for the roundhead protection. The rehabilitation solution is presented in Figure 5 as layout drawing and in Figure 6 as a typical cross section. The rock gradations adopted for the rehabilitation solution are presented in Table 3. It is important to mention that there is quite some experience in Scandinavia using such very large rocks for breakwater construction, and the contractor Pihl has built a number of breakwaters using very large quarry rocks.

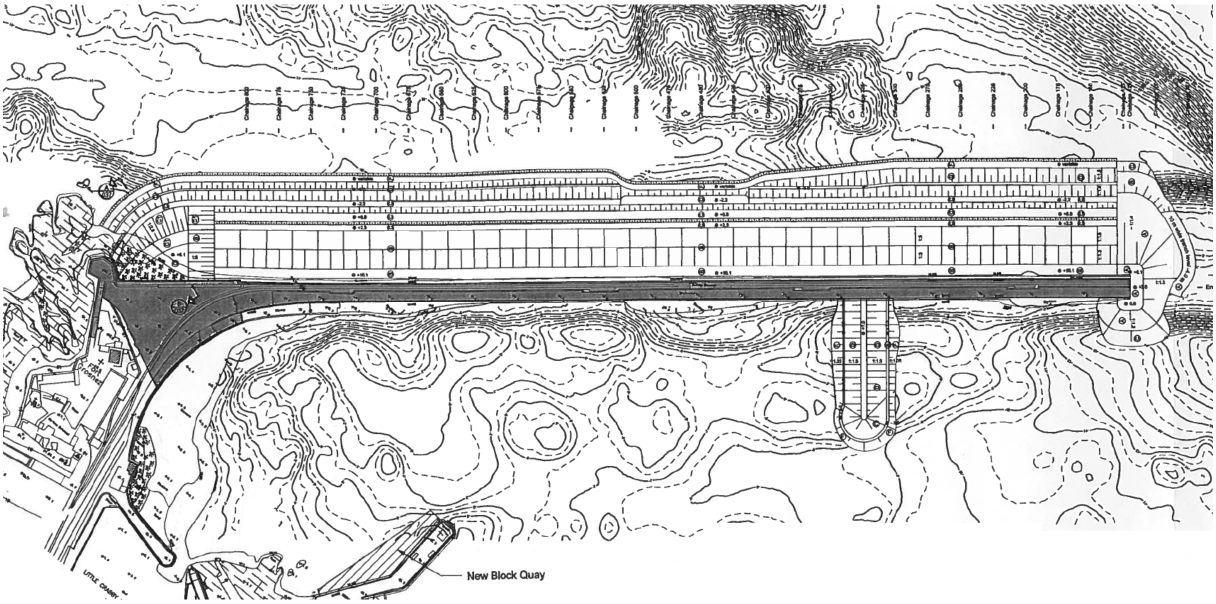

Figure 5 Layout of rehabilitation scheme

Table 3: Rock gradations

Rock class	Range (t)	W_{50} (t)	D_{n50}	D_{15}	D_{85}
			m	M	M
A	20 – 30	24	2.07	1.97	2.20
B	12 – 24	18	1.88	1.61	2.03
C	6 – 14	10	1.55	1.27	1.70
D	1 – 6	2	---	---	---
E	0.2 - 0.5	0.3	---	---	---
F	0.001 - 1.0	----	---	---	---

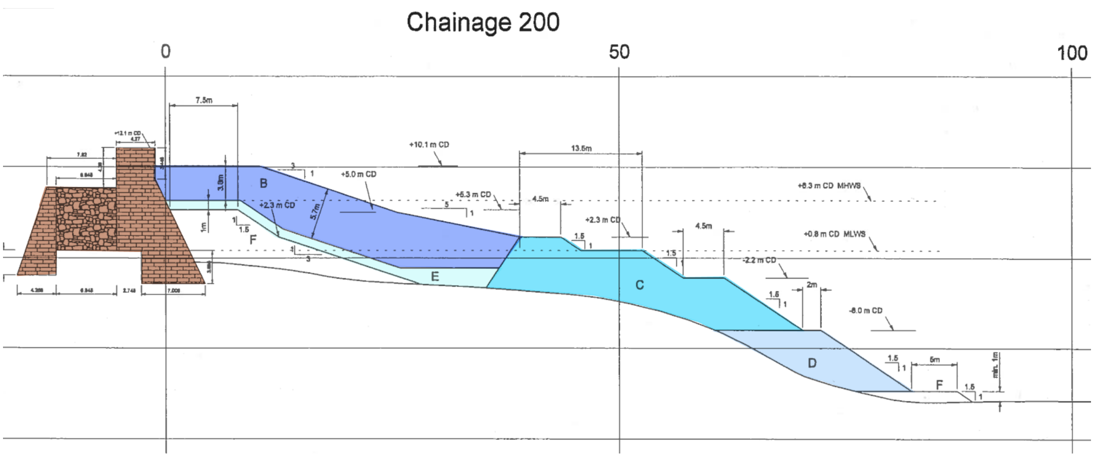

Figure 6 Cross section of rehabilitation solution, at chainage 200

Hydraulic Model Tests

The proposed design was for verification subjected to a few 2D hydraulic flume model tests carried out at DHI. These tests focused on studying the stability of the rubble mound structure, the wave overtopping discharge and measurements of the wave impacts of the front side of the masonry wall. Tests were also carried out for the existing structure, in order to be able to compare results. The tests showed that for the 50 year design wave height, of H_s= 7.2 m and T_m=12.3 s, the rock berm is sufficiently stable.

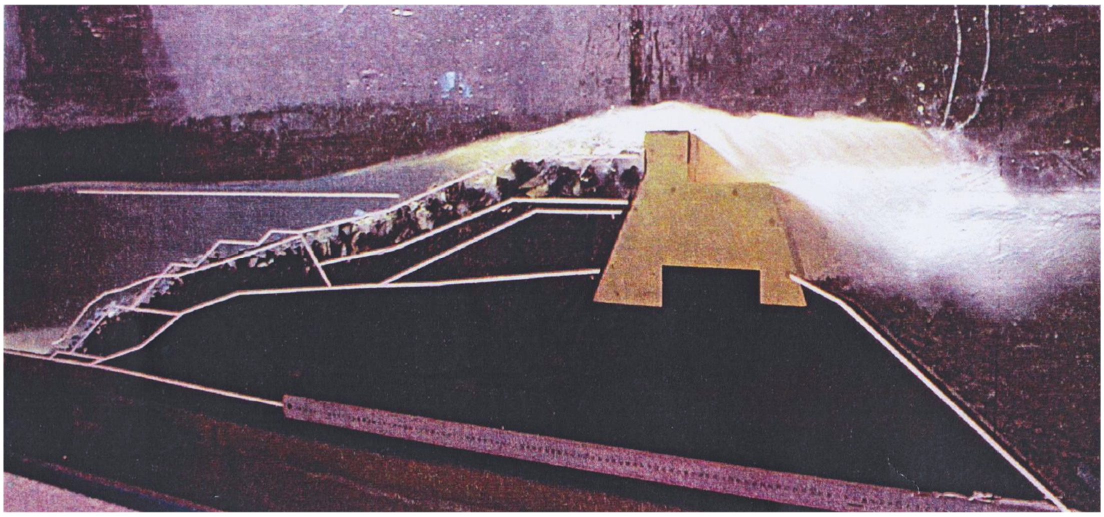

Figure 7 Photo of model with rehabilitation solution and large wave overtopping

The tests further showed a very large reduction in wave overtopping and an even larger reduction in the wave impacts on the front side of the masonry wall. Figure 7 shows a photo from the tests and Table 4 the overtopping results.

Table 4: Model test results on overtopping

Return Period	H_s	WL	Q		Reduction
			Existing	Proposed	
Years	m	mCD (MHWS)	l/s/m	l/s/m	Fold
1	5.2	+6.3	148	4.8	31
50	7.3	+6.3	518	110-156	4

Geotechnical study

The tender geotechnical design of the remediation solution of the breakwater was accomplished according to BS 6031 (1981), BS 8002 (1994), and BS 8004 (1984). The objective of the geotechnical design was to verify that the overall stability of the breakwater has been increased, and that for no part the stability has been decreased when the revetment was placed.

Geotechnical soil profile and parameters

The analysis was carried out using the PLAXIS finite element software and the geometry and soil profile used in the model is derived from the cross section at chainage 200 m. This profile is shown as Figure 8.

The soil types and the assumed characteristic geotechnical parameters are derived from States of Guernsey (2001b). The rock classes in the new revetment, presented in Table 5, are divided into five types B to F, even though all are assumed to have similar geotechnical parameters.

Table 5: Characteristic geotechnical parameters

Classification	Unit weight	Cohesion	Friction angle	Dilatancy angle	Young's modulus	Poisson's ratio	Coefficient of permeability	Interface roughness
	γ'/γ	c'	φ'	Ψ	E	v	k	R
	kN/m³	kPa	º	º	MN/m²	-	m/day	-
Wall	16/26	Linear elastic			100,000	0.3	0.1	0.6
Hearting	12/22	1	50	20	40	0.3	1	1
Mound	12/22	1	42	10	35	0.3	20	0.67
Marine Sand	10/20	1	38	0	20	0.25	1	0.67
Bedrock	17/27	30	50	20	31,000	0.25	0.001	1
Rock B to F	10/20	1	48	15	40	0.3	100	0.67

1. The dilatancy angle ψ is found from the friction angle: ψ ≈ φ - 30º
2. In the marine sand the dilatancy angle is set to zero and the dilatancy angle of the armour rocks is adjusted to 15º
3. The soil properties of the bedrock have been derived from the unconfined compressive tests

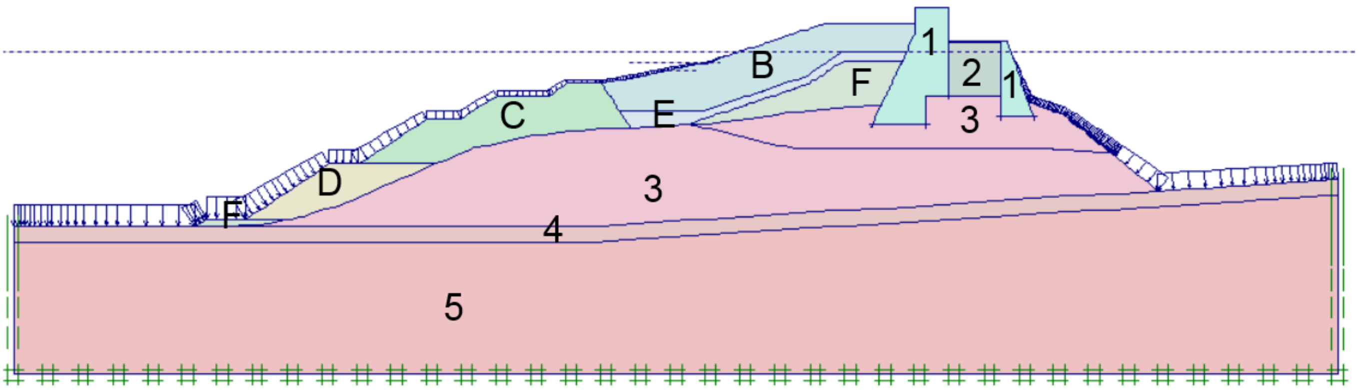

Figure 8 PLAXIS profile at chainage 200 m

Materials
1. Breakwater walls (modelled as blocks)
2. Hearting of the wall
3. Existing rubble mound
4. Marine sand overlying bedrock
5. Bedrock
B to F. Rock classes

PLAXIS calculations

The geotechnical stability of the breakwater was computed in the geotechnical 2D finite element program PLAXIS version 7.2. The model and its boundary conditions are shown in Figure 9. The calculations showed that by introducing the rock berm, the geotechnical stability increased from marginal (existing structure at or close to limit equilibrium) to an acceptable level of safety.

In an ultimate limit state (ULS) the φ-c reduction method used in the PLAXIS analyses. This means that the cohesion and the tangent of the friction angle are reduced in the same proportion:

$$\text{FOS} = \frac{\tan \varphi_{input}}{\tan \varphi_{reduced}} = \frac{c_{input}}{c_{reduced}} \quad (1c)$$

The global safety factor FOS aimed to be achieved at ULS was for a minimum value of 1.2 for an existing structure, Refer BS 8002 (1994).

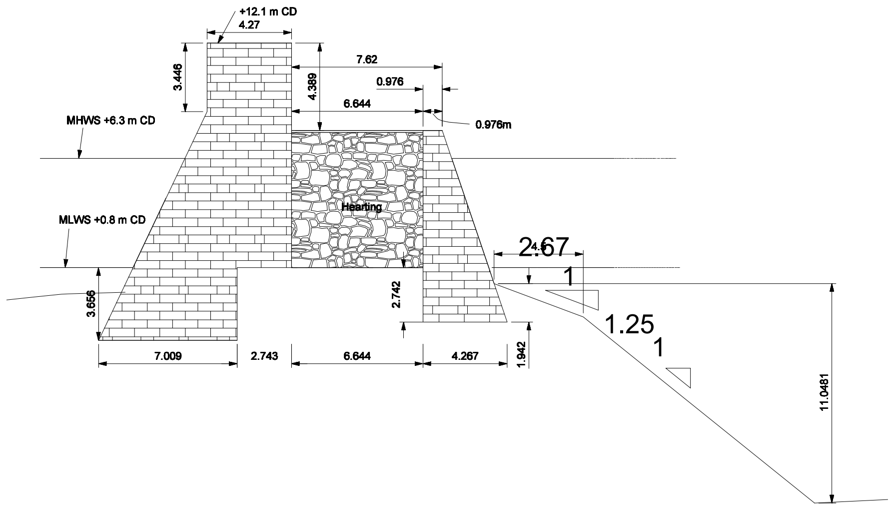

Figure 9 Harbour side mound profile.

Applied loads

The wave impact forces applied to the front of the wall in the geotechnical stability analyses were taken from Danish Hydraulic Institute (2002). The relevant wave impact loads on the front wall are presented in Table 6.

Table 6: Applied wave impact loads on front wall

Test	H_s	T_p	Water level	Pressure at -1.95 m CD	Pressure at +1.15 m CD	Horizontal force
	m	S	m CD	kPa	kPa	kN/m
Existing profile	7.3	16.5	+6.3	137.5	218.0	735.6
After remediation	7.3	16.5	+6.3	42.4	49.9	298.1

The resulting wave pressure distribution on the front of the superstructure from wave impact is presented in Figure 10

Eight PLAXIS ultimate limit state analyses were conducted. In addition an analysis was performed with the wall modelled as a block, in order to see the influence of the contents of the hearting. The main result of the PLAXIS finite element analyses is the global factor of safety FOS. The results are presented in Table 7.

The stability factors for dead weight and dumper truck loads were found to be governed by localized failure zones within the harbour side hearting of the wall and were thus unaffected by the introduction of the rock berm.

Table 7: Global stability safety factor

Calculation type	Existing profile	After remediation
Dead weight	1.23	1.23
Wave impact forces	~0.5	1.17
Dumper truck towards sea	1.23	1.23
Dumper towards harbour	1.03	1.03

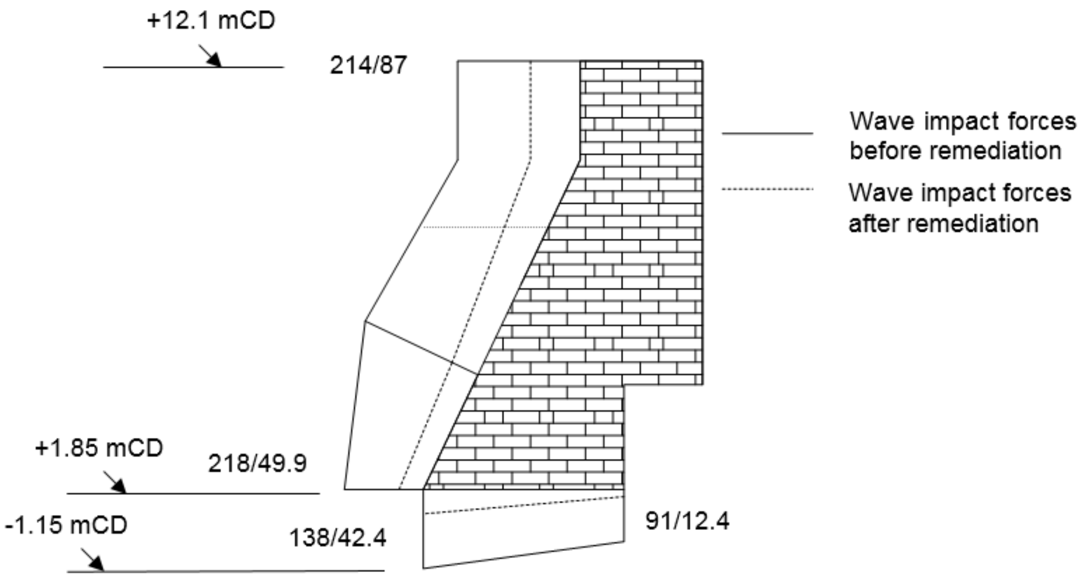

Figure 10 Applied wave impact pressure (in kPa)

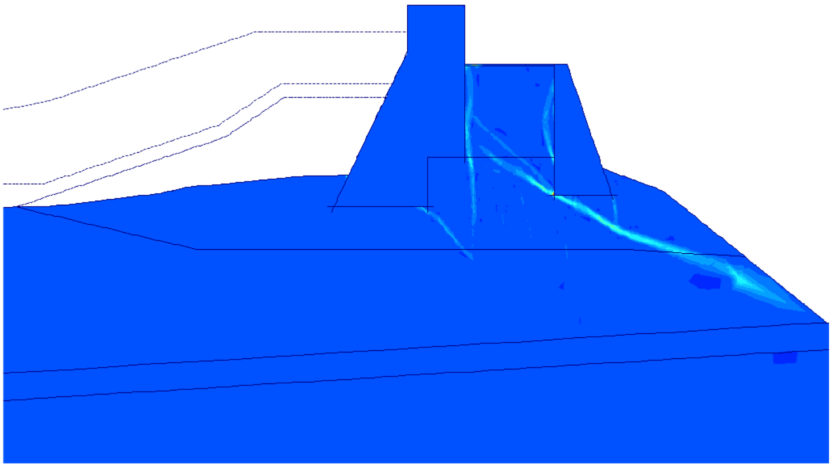

Figure 11 Shear strains indicating imminent failure at 50% wave impact forces before remediation (light blue colour indicates zones with large shear strains)

Applying the wave impact force before remediation the sea wall is developing a geotechnical failure, illustrated by the large shear strains shown in Figure 11. In this calculation only 50% of the expected design wave load can be applied to the model before failure occurs. This geotechnical failure mainly affects the harbour wall. This result may not be in conflict with the observed past behaviour of the breakwater. Here the top of the superstructure has failed several times during extreme storm events. This will cause a reduction in the wave loading on the superstructure, and may be the reason why the remaining part of the superstructure has not failed.

The result in Table 7 shows that the placing of the new revetments dead load does not affect the stability of the superstructure. However, when wave impact forces are applied, the stability of the superstructure is significantly improved after remediation. The global factor FOS is increased from FOS ~ 0.5 to FOS = 1.17 thus (almost) meeting the design intent of FOS ≥ 1.2.

The above PLAXIS modelling has verified the overall stability of the mound and superstructure in an integrated analysis, and thus confirmed the feasibility of the proposed remediation scheme.

Conclusions

Unfortunately, due to lack of sufficient public funds for the project, it was decided not to go forward with the remediation scheme. Consequently, plans for development of a new marina in the basin behind the breakwater appear to have been shelved in 2016 due the unknown future of the breakwater, that would be essential for providing secure and long term wave protection.
The authors are not aware of what has happened in the last about 15 years, except that the development of a new marina has been cancelled due to the unknown future for the breakwater.

In conclusion, the present paper presents a viable, aesthetic and economical rock berm solution for the rehabilitation of the historical Alderney Breakwater. The solution requires further model tests including the roundhead (3D) and detailed design before its implementation.

Acknowledgements

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