S L A P T O N S A N D S B E A C H M A N A G E M E N T P L A N

Defence Baseline Report

Prepared for

Slapton Line Partnership

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SECTION 1

Introduction

1.1 Background and Study Area

This report has been prepared for the Slapton Line Partnership (SLP) and their partners including South Hams District Council, the Environment Agency and Devon County Council, as part of the Slapton Sands Beach Management Plan (BMP). The BMP study area covers the coastline from Torcross in the south, to Strete Gate in the north, as shown in Figure 1.1.

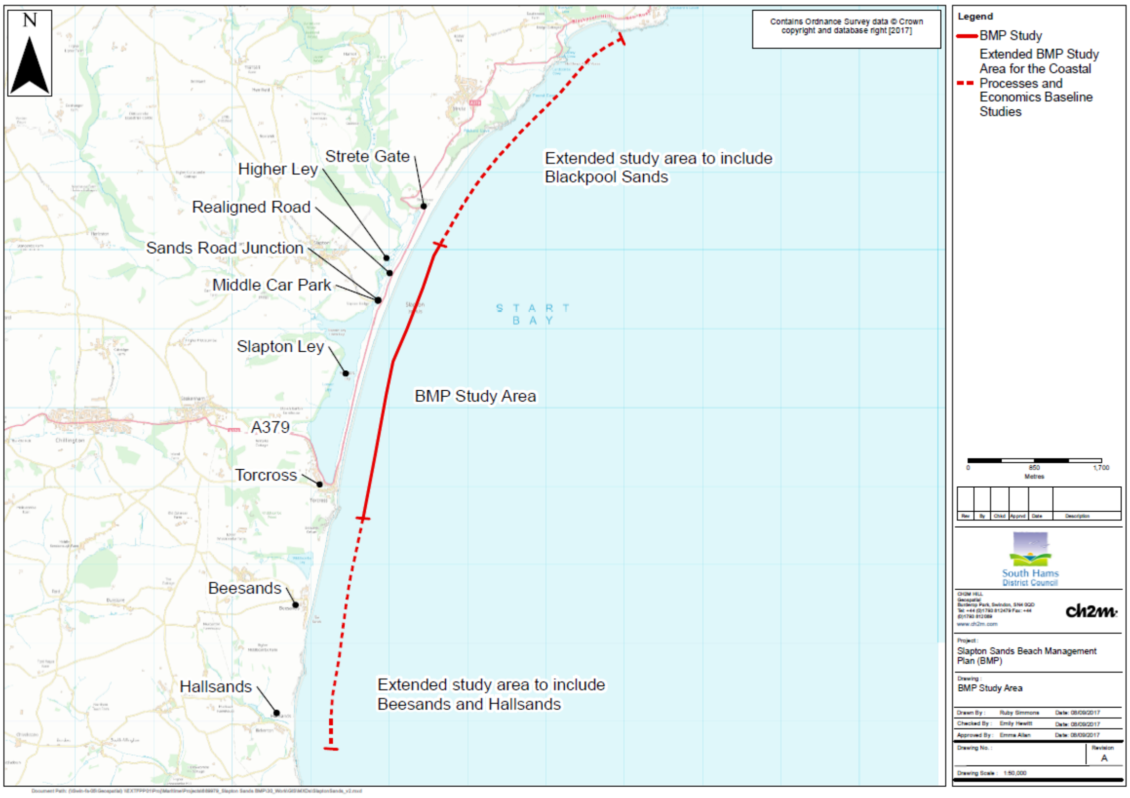


Figure 1.1 Slapton Sands BMP Study Area

Map showing the extent of the BMP study area

1.2 The Basis of this Report

This Defence Baseline Report is a supporting document to the BMP. Studies covering coastal processes, the environment and economics are being undertaken separately and a detailed options appraisal will be completed as part of the BMP process.

This report provides a baseline assessment of the coastal defences located along the Slapton Sands BMP frontage. The purpose of this assessment is to provide information to inform the development of future flood and coastal erosion risk management measures during the options appraisal process. As such, this report includes:

- An outline of the history of defences constructed along the BMP frontage, taken from previous studies and reports that have been reviewed as part of this project (Section 2);
- A new assessment of the current condition of each 'element' of the coastal defences along the frontage (Section 3), completed as part of the present BMP study;
- An assessment of wave overtopping for a range of extreme events, and calculation of the standard of protection provided by the existing coastal defences (Section 4); and
- Conclusions and and recommendations for further investigations (Section 5).

SECTION 2

Defence History at Slapton Sands

This Section provides an outline of the coastal defence history along the BMP frontage, from Torcross to Strete Gate, in order to understand the previous approaches to flood and coastal erosion risk management (FCERM). Section 2.1 provides a chronological summary of the defence history at Slapton Sands, which is supported by photos and technical drawings for the defences and events discussed, referenced within the table and shown in Section 2.2. A summary of defence ownership is presented in Section 2.3.

2.1 Defence History

Table 2-1 provides a chronological summary for the BMP study area, with details of each coastal defence scheme, major storms, and studies/inspections commissioned between 1917 and 2017.

Table 2-1 Chronological summary of coastal defences along the BMP frontage

Year	Construction/ Event	Description	Source
1917	Sheet piling and concrete wall	Sheet piling, capped with concrete top (143m) was constructed north of Torcross.	Slapton Coastal Zone Management (SCZM) (Scott Wilson, 2006)
1979	Rock revetment	A rock revetment (795m) was constructed between Torcross and the middle car park.	
1979-1980	Torcross seawall constructed	A seawall at Torcross was constructed in 1979, with the following principle elements (see **Figure 2.1** for original As-Built drawing): 6m steel sheet piled toe with concrete capping beam (toe level at 3.00m ODN); 5m wide revetment with rock cast in concrete (1 in 2.5 slope); and Recurved seawall (crest level at 6.25m ODN). Natural geomorphological change covered the rock revetment with beach material for approximately 30 years.	Torcross Emergency Work Assessment (EA, 2016a)
Late 1980s	Concrete blockwork installed	Concrete 'Armourflex' blockwork was installed in front of the middle car park (330m).	SCZM (Scott Wilson, 2006)
2000	Torcross seawall modified	The seawall at Torcross was modified by the Environment Agency in 2000.	
Jan 2001	Storm	Severe storms in January 2001 caused a reduction in the crest width of 5m over a length of 1,000m. The erosion undermined a 200m length of the A379, north of the junction with Sands Road, resulting in the temporary closure of the road (see **Figure 2.2**).	
Feb 2001	Assessment of geomorphological impacts in relation to management of the road	A report on the geomorphological impacts in relation to the management of Slapton Road was undertaken after the 2001 storms. The major morphological elements of the barrier beach relevant to the proposed works by DCC and future management were reviewed including the position, elevation and planform.	Pethick (2001)
2002	Road realignment	A 250m section of the A379 adjacent to the Higher Ley had to be reinstated 20m inland. The new two-way length of carriageway replaced a temporary single road which was built after storm damage in 2001 (see **Figure 2.3**). Due to the sensitivity of the site an Environmental Impact Assessment was undertaken.	Planning Application (DCC, 2006). SCZM (Scott Wilson, 2006)

2001	Slapton Line Partnership founded	Organisations with responsibility for the road set up the Slapton Line Partnership to help manage the roads future after the storms in 2001.	-
Jan 2003	Beach redistribution	Beach redistribution and new monument built (see **Figure 2.4**).	South Hams District Council (SHDC)
2005	Bastion management	Bastions were installed along the back of the beach to protect the car park and monument area (see **Figure 2.5**).	SHDC
2006	Slapton Coastal Zone Management report published	The Slapton Line Partnership initiated studies which are summarised in the report entitled Slapton Coastal Zone Management, prepared by Scott Wilson. The report recommends that the A379 should be maintained by a combination of: • Proactive realignment or the northern section of the A379 as soon as funding and permissions were in place; • Reactive realignment of the road at other locations when damage to the road is imminent or has already occurred; and • Localised movement of shingle to provide temporary protection to short lengths of the road or to allow reinstatement of short lengths of the road following damage. The study concluded that the Managed Realignment option would allow the road to be maintained for at least another 30 years. Implicit in this conclusion is the assumption that eventually the effects of sea level rise will make maintenance of the road link economically and environmentally unviable and the road will then be abandoned.	SCZM (Scott Wilson, 2006)
2006	Planning application for A379 realignment made and Environmental Statement prepared to accompany the application.	In 2006, a planning permission was sought to replace and realign two sections of the A379 highway between Slapton and Torcross, considered to be the most vulnerable lengths of road to storm damage. The two lengths are to the east and west of the new road constructed in 2002 (and described above). Location plans of the southern and northern sections of realignment are shown in Figure 2.6 and Figure 2.7. To accompany the planning application, an Environmental Statement was prepared by Atkins on behalf of Devon County Council.	Planning Application (DCC, 2006). A379 – Proposed Carriageway Realignment Environment Statement (Atkins, 2001)
2007	Planning permission granted by DCC	In 2007, Devon County Council gave planning permission for realignment of, what was thought to be, the most vulnerable stretch of the A379, a stretch largely to the north of Slapton Bridge. This permission will not be implemented until the road is subject to sudden damage or damage is seen as imminent.	An Invertebrate Survey of The Slapton Shingle Ridge (Boyce, 2016)
2007	Slapton Ley study published	A project was undertaken to predict the future of the leys if a stable breach was to occur in the future. The key aims of the project were: • To identify the extent of seawater penetration within the leys in the event of a breach; and • To identify measures to ensure the continuation of the freshwater SSSI features that would be lost in the event of a permanent breakdown of the shingle bar.	Royal Haskoning (2007)
2009	Bastion replenishment	Bastion replenishment works were undertaken by SHDC in 2009.	SHDC
Dec 2013	Storm	A large storm event resulted in significant beach erosion, with beach levels dropping by approximately 2-3m from their stable pre-storm profiles. The rock revetment and front face of the steel sheet piles were also exposed.	Torcross Emergency Works Findings Report,

			(Mott MacDonald, 2016)
Feb 2014	Storm	Storms resulted in significant transfer of beach material from locations at the northern end of Slapton Beach to areas at Torcross. Damage to sea-front properties occurred during the storm.	Torcross Emergency Works Findings Report, (Mott MacDonald, 2016)
Feb 2015	Torcross Shingle Recycling and Bastion replenishment	After the 2014 storms, the beach levels in front of the Torcross defences did not recover as quickly as was anticipated. A shingle recycling project was undertaken by SHDC along the Slapton line, including the deposition of shingle from Strete gate at Torcross and the construction of six shingle bastions. 17,041m^3 of material was excavated from Pilchard Cove with: 3,855m^3 deposited at Torcross Point; and 13,186m^3 deposited at six Bastions. Material was collected from within the same coastal cell, at the northern end of Slapton beach (Pilchard Cove). See **Figure 2.8** and **Figure 2.9** to see the General Arrangement drawing and photos of this work.	Torcross Shingle recycling-Technical Briefing Torcross Emergency Works Findings Report, (Mott MacDonald, 2016)
Dec 2015	Slapton Line Economic Valuation report published	The report presented the findings that quantified the present day economic contribution of the Slapton Line Road (A379) that extends north of Torcross. Two aspects of the roads contributions were quantified, including: • The potential effects on local traffic including residents and service providers if the road was temporarily or permanently lost; and • The potential effects on the local visitor economy if the road was temporary or permanent lost.	Slapton Line Economic Valuation (JBA, 2016)
Dec 2015 -Feb 2016	Storm	The beach was severely eroded with material in front of the pile-wall varied between -2m and -1m below the pile capping level (set at +3mAOD). The piles suffered movement and the seawall was overtopped by incident waves. Natural recovery of the beach was recorded shortly after this event.	Torcross Emergency Works Findings Report, (Mott MacDonald, 2016)
Feb 2016	Vulnerability of A379 to Storm Damage across Slapton Ley technical note published	In 2015, following storms, the SLP (comprising Natural England, Devon County Council, South Hams District Council, South Devon AONB, the Environment Agency and the Field Studies Council/Whitley Wildlife Trust) began discussing a range of resilience measures for the A379. In 2016, a vulnerability assessment of the stretch from Torcross to Slapton Bridge was produced. The purpose of the assessment was to enable the SLP Steering Group to achieve a high-level understanding as to the structural damage vulnerability of different sections of the A379. A qualitative analysis of different vulnerability factors on the landward and seaward side of the A379 was provided. The results of the study showed that the most vulnerable sections appeared to be located within the first 700m (north of Torcross) and final 300m (north end of Higher Ley) of the study area.	An Invertebrate Survey of The Slapton Shingle Ridge (Boyce, 2016) South Hams District Council and West Devon Borough Council (2016)
Feb 2016	EA Torcross Defence Inspection	Identification of a new longitudinal opening (5-10mm) over a 110m length at the rear of the concrete roadway slab behind the main Torcross seawall. Inspection concluded that the structure was unstable and that there was a substantial risk of failure within	Torcross Emergency Work

		the short-term if no intervention was taken. It was determined that the site required urgent attention and early action to minimise future remedial costs.	Assessment (EA, 2016a)
Sept 2016	Torcross Emergency Works Investigation report published	Investigation into the structural integrity of the sea defences and assessment of the standard of protection provided. From these investigations, options for remedial works were identified and preference given to implement emergency works. The remedial works suggested are listed below: • Investigate if there are voids behind any section of the sea defence wall or the slipway; • Investigate the structural integrity of the reinforced concrete wave return wall where cracks were showing; • Evaluate the stability of the defence, carrying out investigations as required (e.g. confirmation of substrate at front of the toe, confirmation of pile depth and thickness and soils behind and beneath revetment); • Evaluate if the shingle is integral to the long-term serviceability and condition of the defence, and investigate what feasible measures exist to protect the defence; and • Provide accurate 'as-built' records of the sea defence wall.	Mott MacDonald (2016)
Mar 2016 – Oct 2016	Torcross Emergency Works Findings Report published	This report included background information, data collection, wave assessment, failure assessment, outlined options, assessment of options and conclusions/recommendations. Outline options included: • Do Nothing • Full Rock Armour • Value engineered reduced revetment • Concrete Armour • Sheet Piling • Rock Fillet The report recommended that the short list of potential solutions is discussed in a Workshop meeting to arrive at a preferred option.	Torcross Emergency Works Findings Report, (EA, 2016a)
Nov 2016 – Jan 20017	Torcross Emergency Works	Piling and reinforced concrete capping beam added to the site along the Torcross seawall frontage. The works were completed in March 2017. See **Figure 2.10** and **Figure 2.11** for photos and as-built drawing of the work.	Torcross Weekly Progress Reports, (CH2M HILL, 2017)

2.2 Relevant Historical Photos and Drawings

This Section provides supporting photos and technical drawings to the information provided in Table 2-1. This includes, amongst others, images of the flood defence and coastal erosion structures, before and after photos of the coastline demonstrating the consequences of extreme events at the site, design drawings and plans for realignment of the A379.

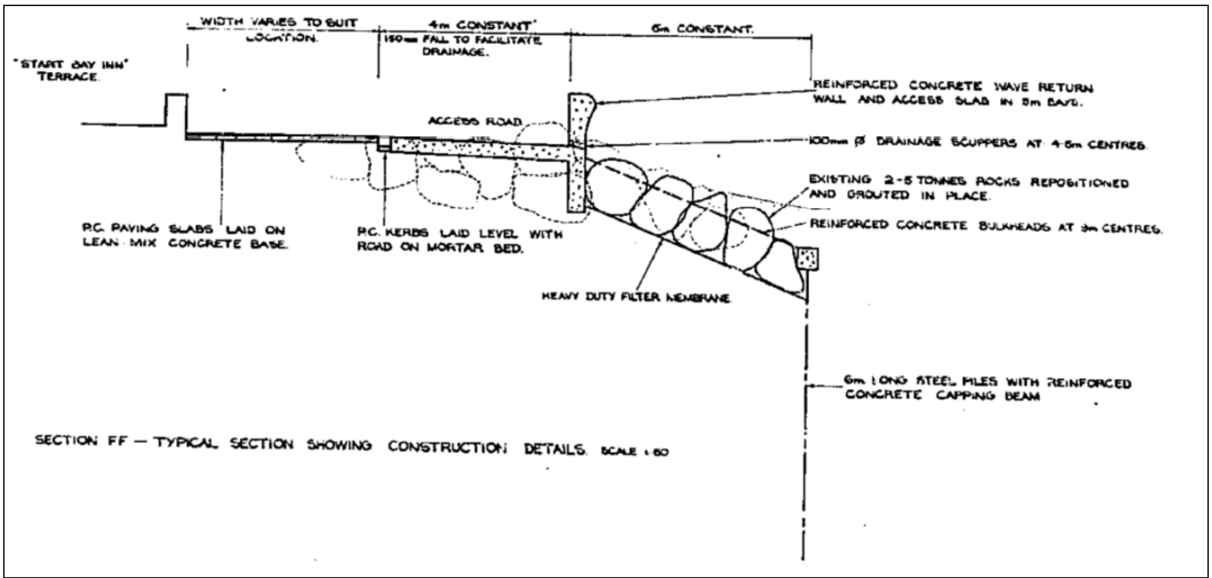


Figure 2.1 1979 as-built drawing of the Torcross seawall (EA, 2016a)

	
Shingle and debris washed onto the A379	Damage and undermining of the A379
	
Damaged ‘armour flex’ blockwork protection	Undermining of historic monument

Figure 2.2 2001 storm damage at various locations along the BMP frontage

Two-lane highway realigned inland of the temporary single lane road

Realigned highway behind the beach

Satellite image of the realigned highway

Figure 2.3 New section of road set back 21m from the existing storm damaged section

Sediment pile in place for redistribution

Monument repositioned further inland

Figure 2.4 2003 beach redistribution and monument placed further inland

Beach material being redistributed over bastion

Figure 2.5 2005 bastion construction

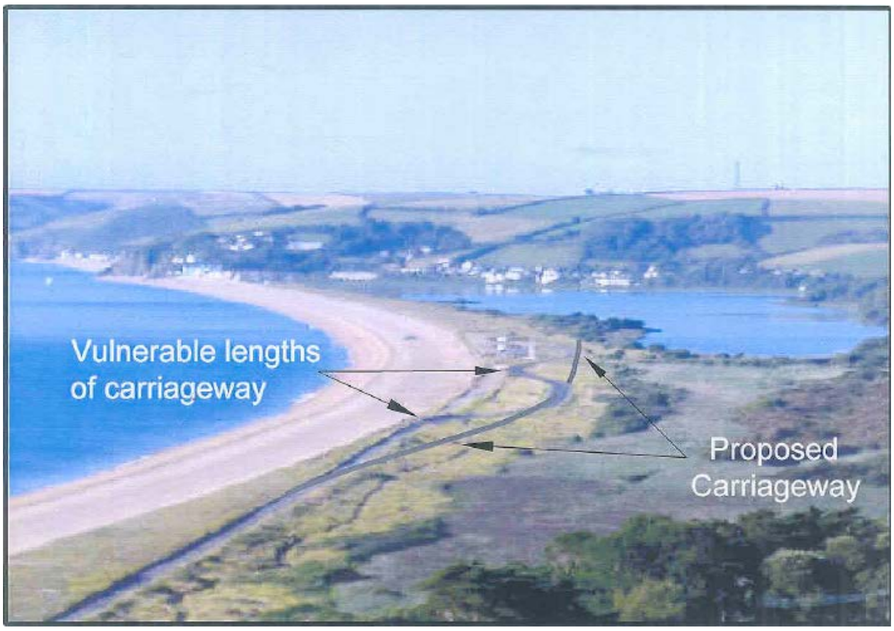
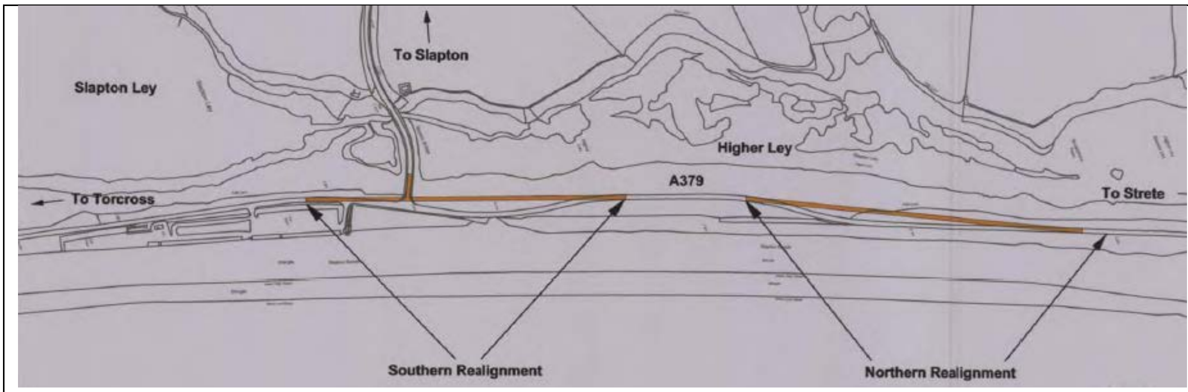
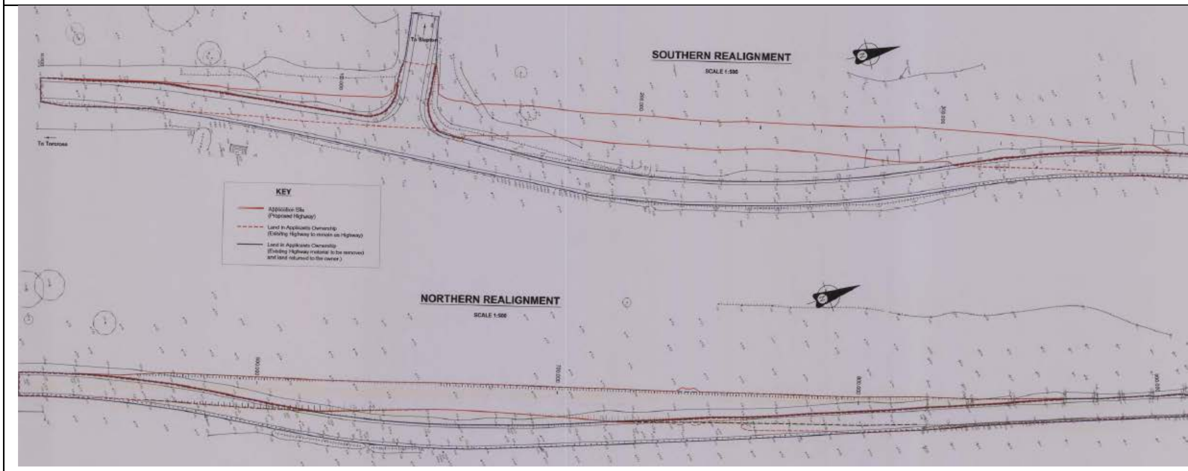


Figure 2.6 Diagram showing location of proposed A379 realignments (DCC, 2006)



Location of the proposed sections of realignment



Proposed southern and northern realignment

Figure 2.7 2006 realignment location proposal

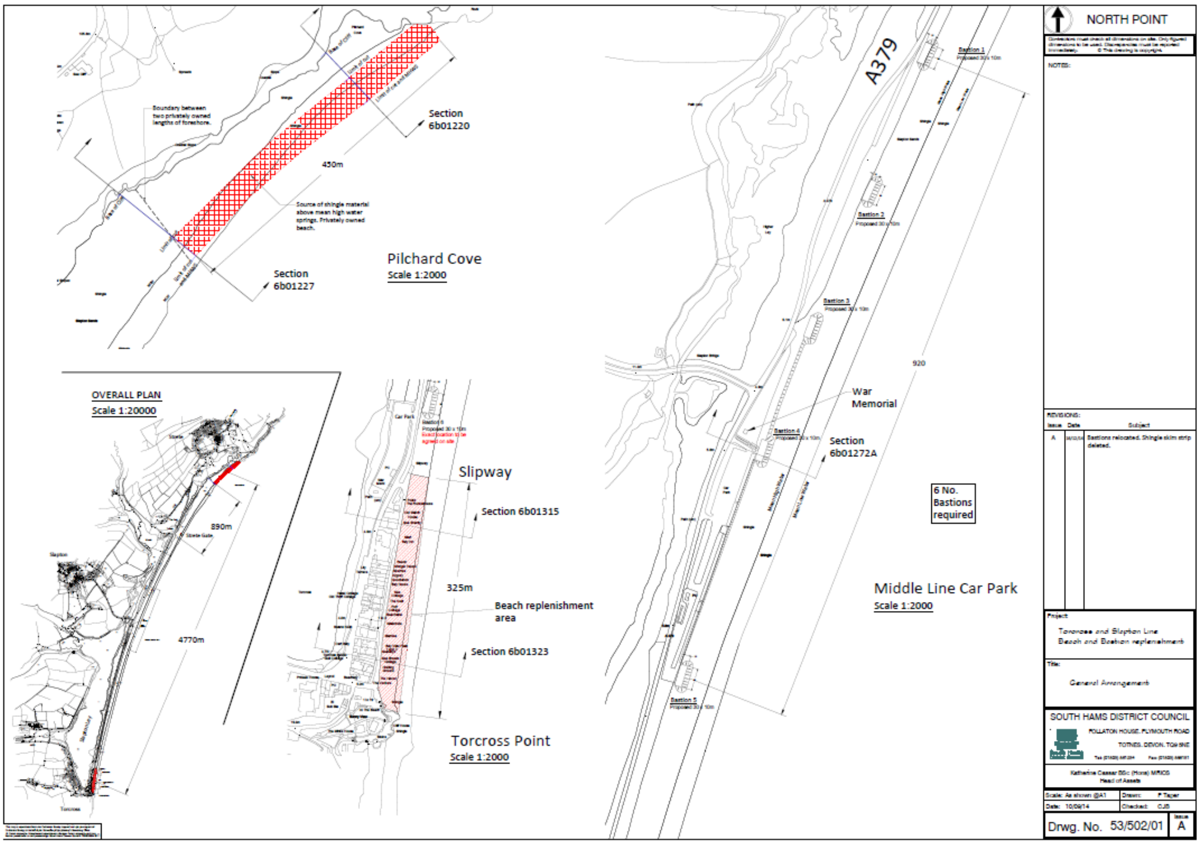


Figure 2.8 2015 Torcross beach and bastion replenishment (SHDC, 2015)

	
Pilchard cove material collection	Truck transporting material south
	
Replenishment of bastion	Distribution of material at Torcross

Figure 2.9 2015 shingle recycling and bastion replenishment photos

	
Sheet Piling undertaken at Torcross	Reinforcement for placement in concrete toe
	
Fixing of the sheet piles	Reinforced concrete capping beam

Figure 2.10 2016/2017 Torcross seawall emergency works photos

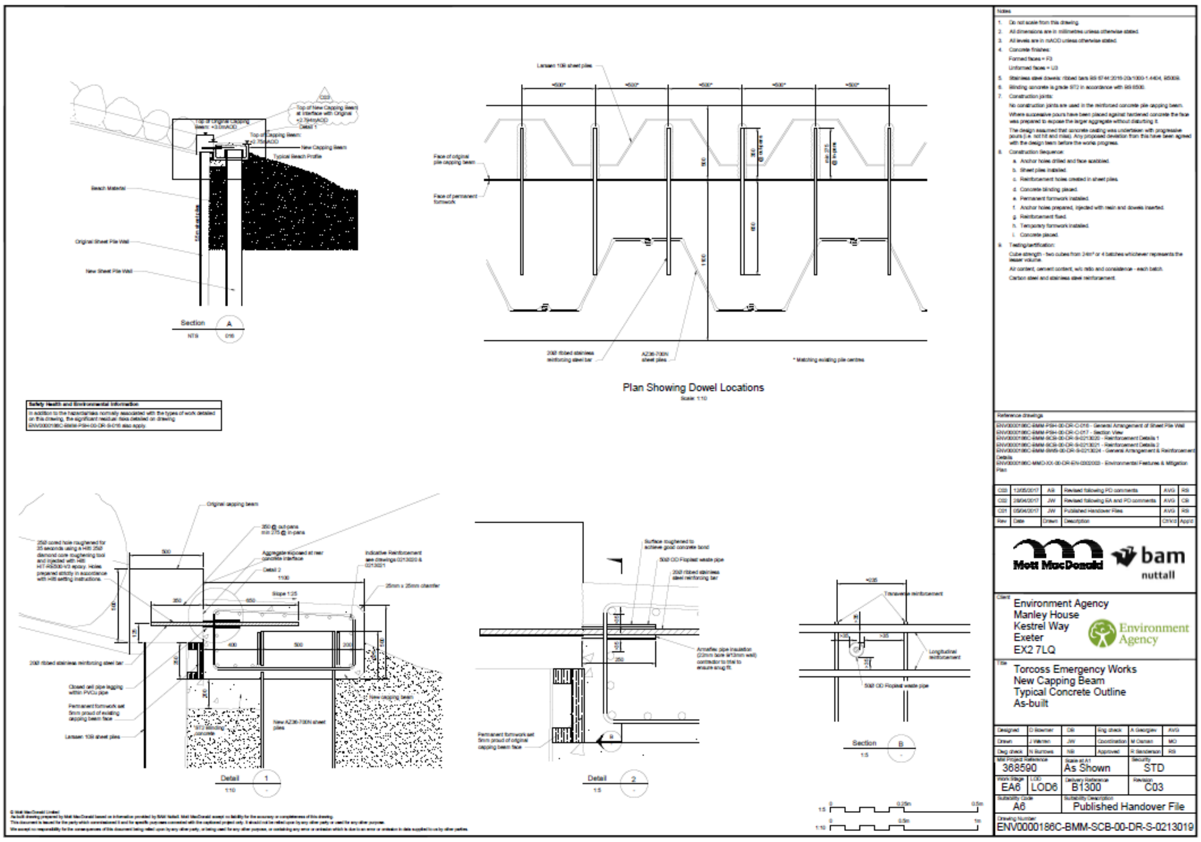


Figure 2.11 2016/2017 Torcross seawall emergency work as-built (Mott MacDonald and BAM, 2017)

2.3 Asset Ownership

The information in Table 2-2 shows each assets length and ownership details, which has been derived from the National Flood and Coastal Defence Database (NFCDD).

Table 2-2 NFCDD Asset Information

Asset No.	Asset Description	Asset length (m)	Ownership/Maintainer
71810	Concrete recurve seawall	163	Environment Agency
71800	Concrete recurve seawall including slipway	151	Environment Agency
71790	Sheet pile wall	30	Local Authority
471780	Concrete recurve seawall	55	Local Authority
71770	Rock armour, undefended section and concrete riprap at carpark.	2092	Local Authority
71765	Undefended length	81	Local Authority
71760	Undefended length (realigned road section)	515	Local Authority
71758	Undefended length	753	Local Authority

Defence Condition Assessment

3.1 Introduction

A visual inspection and condition assessment of the defences along the BMP frontage, between Torcross and Strete Gate was undertaken by CH2M's coastal engineers on the 27th April 2017 to determine their condition and residual life. For the purpose of this assessment, the BMP frontage was divided into ten sections defined by the flood defence or coastal protection measure present, as shown in and listed below.

Figure 3.1 Frontage sections for condition assessment

The sections are numbered from one to ten, and have the following flood defence structure:

1. Concrete seawall;
2. Concrete seawall and revetment;
3. Concrete seawall (Slipway);
4. Sheet pile wall;
5. Concrete seawall;
6. Rock armour protection;
7. 'Armourflex' blockwork;
8. Middle carpark;
9. Beach 1; and
10. Beach 2.

3.2 Methodology

The visual inspection and condition assessment was undertaken in accordance with the Environment Agency's Condition Assessment Manual (CAM) (Environment Agency, 2012), which provides a set of visual indicators and key features for the grading of different types of structures. The CAM provides both general condition grades and condition grades specific to certain coastal defence structures, which are presented in Section 3.2.1 and Section 3.2.2. The Environment Agency's Asset Deterioration Guidance was then used to assess the residual life of each structure in Section 3.2.3.

3.2.1 General Conditions Grades

Table 3-1 describes the five general condition grades that range from 'very good' to 'very poor'.

Table 3-1 General condition grades for structures in accordance with the Environment Agency's CAM

Grade	Rating	Description
1	Very Good	Cosmetic defects that will have no effect on performance
2	Good	Minor defects that will not reduce the overall performance of the asset
3	Fair	Defects that could reduce the performance of the asset
4	Poor	Defects that would significantly reduce the performance of the asset. Further investigation needed
5	Very Poor	Severe defects resulting in complete performance failure

3.2.2 Condition Grades for Defence Structures

The tables provided in this Section (sourced from the CAM, Environment Agency, 2012) show the condition grades (including rating and key features) for the structures that exists along the BMP frontage and as used for the visual inspection and condition assessment between Torcross and Strete Gate. These were used in addition to the general descriptions shown in Table 3-1.

Table 3-2 Concrete seawall grading key features

Grade	Rating	Key Features
1	Very Good	No evidence of structural movement. No Spalling or staining. Minor hairline cracks or honey combing. No loss of backfill material, settlement or undermining. Joints are in good condition with no sealant loss. Beach foreshore appears to be in good condition with no exposure of the structures toe.
2	Good	No evidence of structural movement. No slumping or heave of ground surrounding structure. Minor staining with localised spalling or appearance of small cracks. No settlement or undermining. Minor loss of backfill. Joints in good condition with minimal sealant loss. Some wear to concrete at the base of the structure from wave action and abrasion from shingle.
3	Fair	Minor slumping or heave of ground surrounding the structure. Significant staining. Minor cracking or spalling with exposure of surface reinforcement. Minor loss of backfill. Localised undermining or settlement. Minor cracks or holes in joints due to sealant loss. Lowered beach level in front of the wall.
4	Poor	Minor movement of the structure. Severe slumping or heave of ground surrounding the structure. Minor settlement, undermining or loss of backfill material. Severe cracking or holes in the joints. Severe cracking or spalling with localised areas of main reinforcement. Low beach level, exposure of foundations.
5	Very Poor	Evidence of severe structural movement. Severe settlement, undermining or loss of backfill material. Severe cracking or loss of concrete exposing extensive areas of main reinforcement.

Table 3-3 Rock revetment key features

Grade	Rating	Key Features
1	Very Good	Rocks well packed with no sign of voids. Cross sectional profile consistent along length. No signs of settlement of foundation movement.
2	Good	No signs of significant movement of rock. Cross sectional profile appears consistent along the defence length. Accumulation of material behind defence effectively forms a sacrificial toe increasing performance of the defence. Minor voids present within the rock.
3	Fair	Towards the end of the defence some of the rock has been displaced causing a slump of rocks above onto the beach. Presence of voids between rock. No sign of undermining or damage to geotextile layer below rock.
4	Poor	Minimal quantity of rock provides little protection. Rock sparse in place. Some rock has moved seawards away from the main defence. No consistency in profile. There appears to be no geotextile below the rock. Scour at base of revetment is causing rock to sink into the beach.
5	Very Poor	Minimal rock of suitable size, suitable rock is sparsely placed. Presence of smaller rock within the revetment can move under wave action and increase the potential for erosion. Evidence of wave attack to land backing the defence.

Table 3-4 Concrete revetment key features

Grade	Rating	Key Features
1	Very Good	No signs of cracking to the concrete blocks. No sign of vertical or lateral movement of the revetment. No exposure of the defence toe.
2	Good	Minor loss of joint material between concrete blocks. No signs of vertical or lateral movement, no differential settlement of blocks. No damage to pavement topping defence. Varying beach levels but minimal sign of revetment toe beam.
3	Fair	Signs of minor loss of joint material between concrete panels. Minor spalling to concrete. Beach levels appear low and toe beam is exposed.
4	Poor	Loss of joint material. Significant differential settlement and movement of concrete panels. Movement of wave return at top of revetment.
5	Very Poor	Complete displacement of and breakup of concrete panels. Loss of fill material. Exposure of walkway/road at the rear of the defence.

Table 3-5 Beach key features

Grade	Rating	Key Features
1	Very Good	Wide substantial slope, backshore and crest with no evidence or erosion. No cliffing. Stable beach profile with minimal changes between inspection periods. Established vegetation possibly with young plant growth. Minor foreign objects may be present but causing no scour or instability.
2	Good	Shallow and wide slope with minor or localised erosion. Beach profile fluctuates seasonally with profile recovery under beach building conditions. Backshore remains wide and high with strand line on mid/lower beach. Minor/localised erosion of backshore or crest indicated by cliffing. Established vegetation. Minor localised scour due to presence of foreign objects with no effect on stability.
3	Fair	Minor or localised erosion of slope or toe resulting in reduction of slope width. Minor erosion of backshore or crest indicated by cliffing. Strand line high on backshore indicates reduced backshore or crest width. Localised areas of vegetation. Minor foreign objects present with possible localised effects on stability associated with minor scour.
4	Poor	Sustained and prolonged erosion of beach slope, toe, backshore or crest. Extensive cliffing. Strand line is high on backshore indicating frequent inundation. If seawall is present toe will be exposed. No bedrock exposed. Significant and extensive damage to vegetation. Severe

		scour around foreign objects. No significant beach crest. Spring tides allow direct wave attack to the base of the cliff.
5	Very Poor	Sustained and prolonged erosion of beach slope and toe with significantly lowered beach profile. Severe and extensive cliffing. Bedrock may be exposed. Strand line occurs high on backshore. Evidence of significant overtopping exhibited by sediment on landward side of the crest, runnels from overtopping water and damage to backshore plants from over washing. Complete loss of vegetation. Severe foreign objects present resulting in significant scour. Beach volume depleted resulting in loss of the beach crest and direct wave attack to land at the rear.

3.2.3 Residual Life Estimation

In addition to the CAM, the Environment Agency has produced guidance on how to calculate the residual life of various flood defence assets (Environment Agency, 2013). The guidance has a series of models which can be used to predict the progression of an asset's condition through the five condition grades for various asset class/material combinations. This guidance was used to calculate the residual life of the assets within the Slapton Sands BMP frontage between Torcross and Strete Gate.

The models incorporate three different maintenance regimes and three deterioration categories which are defined in Table 3-6 and Table 3-7.

All the assets were assessed as falling within the medium deterioration rate category, however the maintenance regimes varied.

Each asset has been assessed to identify the number of years until significantly reduced performance (time to transition to Condition Grade 4) and the number of years until complete performance failure is reached (time to transition to Condition Grade 5).

Table 3-6 Outline deterioration categories from the Asset Deterioration Guidance (EA, 2013)

Deterioration Categories	
Slowest	Arising from a sheltered location and/or high quality materials and construction, well-designed asset.
Medium	Considered a typical rate providing a mid-range value representing an average situation, with assets being neither exposed nor sheltered.
Fastest	Arising from an exposed location and/or poor quality materials/construction/design.

Table 3-7 Outline maintenance regimes from the Asset Deterioration Guidance (EA, 2013)

Maintenance Regimes	
Regime 1 Low (do minimum) maintenance	Inspection and H&S repair (annually)
Regime 2 Medium maintenance regime	Inspection and H&S repair (annually) Maintenance activities as proposed in the Environment Agency Maintenance Standards (Environment Agency 2010 and Environment Agency 2012, Appendix B) for maintaining at target CG 3 (Note: The maintenance standards will also pick up minor reactive repairs)
Regime 3 High maintenance regime	Inspection and H&S repair (annually) Maintenance activities as proposed in the Environment Agency Maintenance Standards for maintaining at target CG 2 (Note: The maintenance standards will also pick up minor reactive repairs)

3.3 Section 1: Concrete Seawall 1

Asset Description: Section 1 is a concrete seawall which extends 20 meters north along the beach from the head rock at Torcross Point in front of the Torcross Hotel. The wall is a near-vertical, 2.5m-high, concrete recurve wave wall, with drainage holes located approximately every 4 meters.


Concrete seawall with access steps


Southern end embedded into bedrock/cliff


Middle section showing rust staining and surface cracking


Northern end with concrete access steps

Figure 3.2 Visual assessment of the southern seawall at Torcross

Condition Description: The wall has minor rust staining and surface cracking with one minor void (approx. 100mm diameter) and patch repairs apparent at various location. However, there was no evidence of structural movement or slumping.

Condition Grade: Grade 2 – GOOD

Residual Life: For the seawall, the best estimate for a significant reduction in performance in accordance with the asset deterioration guidance (Environment Agency, 2013) is 40 years. The best estimate for complete performance failure of the asset is 55 years. Table 3-8 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-8 Section 1: Seawall estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	30 years	40 years
Medium	**40 years**	**55 years**
High	50 years	70 years

3.4 Section 2: Concrete Seawall 2

Asset Description: This section is the main flood defence, providing protection to greater than 20 properties in Torcross located (at the southern end of the Slapton Ley). The defence is approximately 300m long and is composed of an upper concrete recurve wall and lower sloped concrete and rock revetment. The defence is separated into several bays, each approximately 5m wide. The rock armour within the revetment varied in size, but was estimated to be 3-6t.

Concrete recurve wall with sloped revetment providing a flood defence to properties at Torcross

Typical section of wall showing rocks embedded in the revetment

Green paint indicating the location of patch repairs

Concrete revetment with no rocks

Figure 3.3 Visual assessment of the northern seawall at Torcross

Condition Description: The wall has minor surface wear, very minor surface cracking and rust staining in places. The joints between the sections are well maintained with a few locations having new replacement sealant. The toe was covered with beach material at the time of the inspection so was not assessed, however work has been undertaken very recently to repair the toe and it is therefore assumed to be in good condition.

Condition Grade: Grade 2 – GOOD

Residual Life: This seawall was in the same condition as the adjacent wall to the south. The best estimate for a significant reduction in the performance of the asset is 40 years, whilst for complete performance failure the best estimate is 55 years. Table 3-9 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-9 Section 2: Seawall estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	30 years	40 years
Medium	**40 years**	**55 years**
High	50 years	70 years

3.5 Section 3: Concrete Seawall 3

Asset Description: The concrete slipway is located at the northern end of the Torcross seawall, has an approximately 10m long slope. The structure has rock armour protection along the seaward edge of the slipway, and a vertical concrete seawall along the landward edge of the slipway.

Slipway concrete wall

View looking south towards the slipway and wall

Evidence of concrete cracking

View looking north towards the adjacent sheet pile wall

Figure 3.4 Visual assessment of the slipway wall at Torcross

Condition Description: There is some rust staining on the seaward concrete but signs of good maintenance and replacement of sealant in joints. The old concrete seawall on the landward side of the slipway is showing signs of cracking and loss of structural concrete at the crest however there are no signs of displacement.

Condition Grade: Grade 3 – FAIR

Residual Life: The concrete wall at the Torcross slipway was assessed as being Grade 3 - Fair condition. The best estimate for a significant reduction in the performance of the asset is 15 years, whilst for complete performance failure the best estimate is 30 years. Table 3-10 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-10 Section 3: Slipway wall estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	10 years	20 years
Medium	**15 years**	**30 years**
High	20 years	40 years

3.6 Section 4: Steel Sheet Piling

Asset Description: A steel sheet pile wall with rock/concrete crest protection extends north from the slipway. Rock armour is intermittently placed at the toe of the structure. The sheet pile wall is approximately 50m long and consists of U-type sheet piles with bays approximately 1.2m wide and 400mm deep.


Sheet pile wall north of the slipway


Sheet pile with 2m ranging pole for scale


Sheet pile with concrete back fill

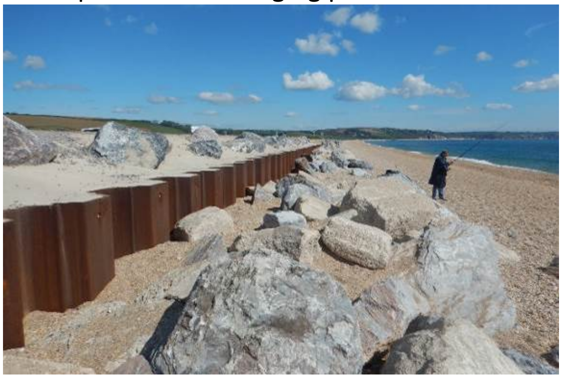

Extend of rock armour fronting the piled wall

Figure 3.5 Visual assessment of the steel sheet pile repair works

Condition Description: There is some surface corrosion of the piles, but no signs of significant damage or evidence of substantial saline penetration. The rock at the toe of the sheet pile wall is not consistently placed and includes some sections of concrete rubble. The concrete behind the sheet piles appears to be new and rocks have been placed in the concrete to provide added protection.

Condition Grade: Grade 2 – GOOD

Residual Life: The sheet pile wall north of Torcross was assessed as being Grade 2- Good condition. The best estimate for a significant reduction in the performance of the asset is 35 years, whilst for complete performance failure the best estimate is 45 years. Table 3-11 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-11 Section 4: Sheet pile wall estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	20 years	30 years
Medium	**35 years**	**45 years**
High	40 years	50 years

3.7 Section 5: Concrete Seawall 4

Asset Description: This defence is an old concrete recurve seawall extending north from the steel sheet pile defence for approximately 50m. At the toe of the wall there was a small amount of irregularly placed rock armour.


Concrete wall north of the sheet pile wall


Major horizontal crack along top of the wall


Typical vertical crack found at joints in wall


Evidence of patch repairs to northern end of wall

Figure 3.6 Visual assessment of the concrete recurve wall

Condition Description: The seawall had significant signs of vertical and lateral cracking (particularly along the bull-nose), large areas of spalling and minor displacement, however the crest level was relatively consistent with no slumping observed. The rock armour at the toe of the wall was inconsistently placed, providing uncertain protection to the structure.

Condition Grade: Grade 4 – POOR

Residual Life: The concrete wall north of the steel sheet pile wall was assessed as being Grade 4-Poor condition. It has therefore reached the point at which there is a significant reduction in the performance of the asset, and has an estimated 15 years before it reaches complete performance failure. Table 3-12 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-12 Section 5: Concrete recurve seawall estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	0 years	10 years
Medium	**0 years**	**15 years**
High	0 years	20 years

3.8 Section 6: Rock Armour Protection

Asset Description: Section 6 includes an approximately 700m length of rock armour which makes up the primary defence against erosion for the A379. Rock has been placed at the back of the beach to reduce overtopping and erosion of the road.

Rock protection looking north

Erosion of the embankment behind the rocks

Voids present behind misplaced rocks

Further erosion of the bank

Figure 3.7 Visual assessment of the rock protection north of Torcross

Condition Description: The rock armour has an inconsistent profile, with some displaced rocks and voids. Bank erosion was evident behind the rock armour in some areas. The defences were in very steep in places, with evidence that fine material had previously been eroded from the bank. It was unclear whether this section of rock armour was part of the emergency works to protect the road or part of a more permanent flood defence scheme.

Condition Grade: Grade 3 – FAIR

Residual Life: For the rock protection, adjacent to the main link road, the best estimate for a significant reduction in the performance of the asset is 20 years, whilst complete performance failure is 30 years. Table 3-13 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-13 Section 6: Rock protection estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	22 years	31 years
Medium	**20 years**	**30 years**
High	40 years	60 years

3.9 Section 7: Armour Flex Blockwork Erosion Protection

Asset Description: This section includes 50m of premade 'Armourflex' concrete units held in place by nylon/chord, overlying a geotextile and general bank material. The structure created a very steeply sloped defence

'Armourflex' blockwork

Large area exposed due to washout of blocks

Slumping of the defence with nylon cords visible

Erosion has exposed rebar peg holding nylon cords

Figure 3.8 Visual assessment of the erosion protection work south of Slapton Sands middle carpark

Condition Description: The nylon/chord was broken in several locations, the underlying geotextile was damaged and torn, and there was surface slumping of the structure. In areas of the worst damage there is complete failure of the structure, with undermining, washout of the backfill, and landward erosion.

Condition Grade: Grade 5 – VERY POOR

Residual Life: The embankment protection south of the car park was assessed as being Grade 5 – Very Poor. It has therefore already reached complete performance failure. The zero values within Table 3-14 highlight that the embankment has already failed.

Table 3-14 Section 7: Erosion protection (Permeable Revetment) estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	0 years	0 years
Medium	**0 years**	**0 years**
High	0 years	0 years

3.10 Section 8: Middle Car Park Embankment

Asset Description: This section includes the Slapton Sands middle carpark which has no remaining flood defence structures. The car park is approximately 200m long and located directly adjacent to the shingle beach.

Erosion/cliffing of the beach at the car park

Cracking of tarmac within the car park due to undermining

Cracking of the carpark as result of erosion

Remains of one of the failed bastions

Figure 3.9 Visual assessment of the middle car park

Condition Description: The tarmac of the parking area and underlying substrate has been undermined and eroded, leading to loss of parking capacity. Fresh breaks in the tarmac surface indicated that this was an ongoing process. The beach fronting the car park was shallower and wider than the beach south of this area.

Condition Grade: Grade 5 – VERY POOR

Residual Life: The embankment at the carpark backing onto the beach was assessed as being Grade 5 – Very Poor. It has therefore already reached complete performance failure. The zero values within Table 3-15 highlight that the embankment has failed.

Table 3-15 Section 8: Tarmac/embankment estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	0 years	0 years
Medium	**0 years**	**0 years**
High	0 years	0 years

3.11 Section 9: Beach 1

Asset Description: Section 9 includes the section of beach between the middle carpark and the northern bend in the A379. The beach at this location was noticeably wider than previous beach sections, and at points was approximately a metre below the road level.

Overview of section

Large section of collapsed material

Evidence of recent erosion

Vegetation falling away from the backshore

Figure 3.10 Visual assessment of beach section 1

Condition Description: A wide beach with substantial erosion of the backshore and cliffing evident. There was some damage to vegetation where cliffing had occurred, and the distance between the beach crest and the road varied. Historical data indicated a fluctuation of the beach levels in this section. Due to the road, the beach is not able to roll back as it might do if the road were not in place, therefore erosion rates and the impacts of sea level rise should be considered for this beach area.

Condition Grade: Grade 3 – FAIR

Residual Life: This section of beach was assessed as being Grade 3 – Fair, therefore the best estimate for significant reduction in the performance of the asset is 20 years whilst the complete performance failure is 45 years. Table 3-16 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-16 Section 9: Beach 1 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	12 years	22 years
Medium	**20 years**	**45 years**
High	35 years	65 years

3.12 Section 10: Beach 2

Asset Description: Section 10 includes the northern section of beach at Strete Gate, where the A379 transitions inland. The beach is composed of finer gravel material and forms a shallower beach with a wide flat crest area.

Wide and shallow beach

Vegetation on the back shore

Figure 3.11 Visual assessment of beach section 2

Condition Description: The beach is wider at this location than the beach to the south, has a shallow beach slope and a wide beach crest, with much more space between the beach and the road. The beach profile fluctuates seasonally, however the backshore remains wide with established vegetation and only minor localised erosion.

Condition Grade: Grade 2 – GOOD

Residual Life: The best estimate for significant reduction in the performance of the asset is 34 years, whilst complete performance failure is 59 years. Table 3-17 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 3-17 Section 10: Beach 2 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	16 years	26 years
Medium	**34 years**	**59 years**
High	70 years	100 years

3.13 Summary of Defence Condition

Table 3-18 summarises the assessed defence condition and residual life of each coastal defence element along the ten sections of frontage described in Sections 3.3 – 3.12.

Table 3-18 Summary of condition grade and residual life assessment for each coastal defence element

Frontage	Defence element	Condition Grade	Residual Life Estimate to Significantly Reduced Performance	Residual Life Estimate to Complete Performance Failure
1	Concrete seawall 1	2 (Good)	40 years	55 years
2	Concrete seawall 2	2 (Good)	40 years	55 years
3	Concrete seawall 3	3 (Fair)	15 years	30 years
4	Steel sheet piles	2 (Good)	35 years	45 years
5	Concrete seawall 4	4 (Poor)	0 years	15 years
6	Rock armour protection	3 (Fair)	20 years	30 years
7	'Armourflex' blockwork	5 (Very Poor)	0 years	0 years
8	Middle carpark embankment	5 (Very Poor)	0 years	0 years
9	Beach 1	3 (Fair)	35 years	45 years
10	Beach 2	2 (Good)	34 years	59 years

Overtopping Analysis

4.1 Introduction

To better understand the flood/breach risk to Torcross and the beach fronting the A379 road, it is necessary to know the levels of wave overtopping that can occur along the BMP frontage. Following a review of existing data and information for the defence history (refer to Section 2), it is apparent that there is little existing information regarding the standard of protection against wave overtopping for the BMP frontage. However, at the time of writing this report new wave overtopping analysis was being completed by JBA on behalf of the Environment Agency for the State of the Nation (SoN) project, but the results for the BMP study area were not programmed in-line with the development of the BMP. Therefore, ahead of the release of the JBA work, and at the request of the SLP, new overtopping analysis was completed for the present study, with a view to compare the results of the new analysis and JBA work as it became available. The JBA work has since been completed and a comparison of results is presented in Section 4.4

Wave overtopping analysis was completed to determine the level of protection that the current defences defence afforded to wave overtopping in 2017, and how it may change in the future.

In order to assess the expected overtopping discharge along the varying coastline, the study area was separated into the same ten sections defined for the defence visual inspection condition assessment (refer to 3.1). A representative beach profile was selected in the middle of each section (see Figure 4.1).

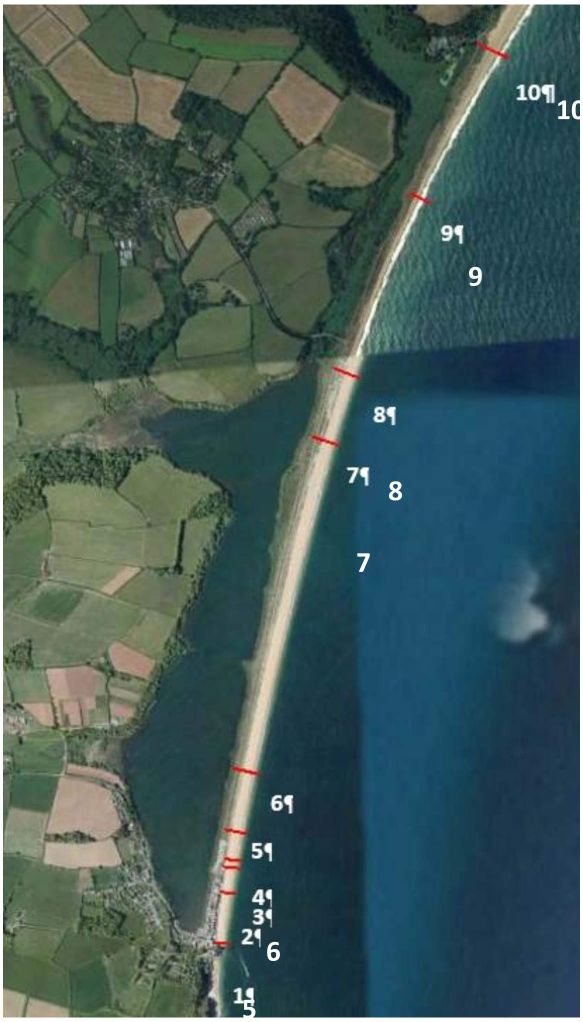


Figure 4.1 Beach profile locations and table summarising the overtopping analysis for each profile

4.2 Methodology

The overtopping analysis has been undertaken in accordance with best-practice guidance contained in the Wave Overtopping of Sea Defences and Related Structures: Assessment Manual 2 (Environment Agency, 2016b).

4.2.1 Overtopping Tolerances

The guidance includes various analytical approaches depending on the type of defence under consideration, and advises different overtopping tolerances depending on the hazard type, structure, and the significant wave height. Table 4-1 highlights the analytical approach taken for each section of defence, whilst Figure 4.2 and Figure 4.3 indicate the overtopping limits considered appropriate for this analysis.

Table 4-1 Summary of the overtopping analysis used for each defence profile

Frontage	CCO Profile	Defence description	EurOtop 2 Eqn.
1	6b01320	Concrete seawall at crest of shingle beach	Eqn. 5.12
2	6b01316	Concrete seawall and revetment at crest of shingle beach	Eqn. 5.12
3	6b01314	Concrete seawall at crest of shingle beach	Eqn. 5.12
4	6b01313	Steel sheet pile at crest of shingle beach	Eqn. 5.12
5	6b01311	Concrete seawall at crest of shingle beach	Eqn. 5.12
6	6b01306	Rock protection at crest of shingle beach	Eqn. 6.21
7	6b01278	'Armourflex' blockwork	Eqn. 6.21
8	6b01273	Carpark embankment at crest of shingle beach	Eqn. 6.21
9	6b01258	Embankment at crest of shingle beach	Eqn. 6.21
10	6b01246	Wide gravel beach	Eqn. 5.12

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 600 600 600 No limit
Cars on seawall / dike crest, or railway close behind crest H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m	 <5 10-20 <75	 2000 2000 2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Figure 4.2 Limits for overtopping for people and vehicles (Environment Agency, 2016b)

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Rubble mound breakwaters; H_{m0} > 5 m; no damage	1	2,000-3,000
Rubble mound breakwaters; H_{m0} > 5 m; rear side designed for wave overtopping	5-10	10,000-20,000
Grass covered crest and landward slope; maintained and closed grass cover; H_{m0} = 1 – 3 m	5	2,000-3,000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; H_{m0} = 0.5 – 3 m	0.1	500
Grass covered crest and landward slope; H_{m0} < 1 m	5-10	500
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit	No limit

Figure 4.3 Limits for overtopping for structural design of breakwaters, seawalls, dikes and dams (Environment Agency, 2016b)

4.2.2 Water Levels and Wave Conditions

Water level and wave data was adopted from the Environment Agency's State of the Nation (SoN) Flood Risk Analysis, Coastal Boundary Conditions (Environment Agency, 2017). The closest SoN data point to Torcross is node 919 (shown in Figure 4.4) which was approximately 200m offshore, and this wave data was applied to the entire study area. The base date of the data is 2014.

Figure 4.4 SoN wave data locations close to Slapton Sands

The SoN data associated to return periods included only wave heights and water levels, and examined return periods of 20, 50, 100, 200, 500, and 1,000 years. Several assumptions were required to establish the full set of input conditions required for overtopping analysis within the study area.

The angle of wave approach for the incident waves was assumed to be zero, e.g. it is assumed that waves approach the coast normal to the coastal defence. This provided a conservative estimate of wave overtopping discharge as the influence factor for oblique wave attack (gb) used in the overtopping analysis is set to one for this condition.

Waves generated from the west or southwest are unlikely to be refracted sufficiently around the Start Point headland to approach the study area normal to the flood defences. Oblique approach can significantly reduce the associate wave overtopping volumes.

A further assumption was required regarding peak wave period, which was established through adoption of Equation 1, proposed by Boccotti (2000).

$$T_p = 8.5\pi\sqrt{\frac{H_s}{4g}}$$

Equation 1 Relationship between peak wave period and significant wave height (Boccotti, 2000).

The relationship was established for typical mid-range deep water conditions, therefore there is uncertainty over how applicable the calculated peak wave period is for nearshore analysis. However, this approach is considered reasonable as the longest fetch available for waves approaching normal to the flood defences is across the English Channel, limiting the wave period that can develop.

The SoN data included multiple joint probability combinations for each return period. To avoid unnecessary analysis, a subset of ten joint probability combinations were selected to provide an even distribution between large wave heights with low water levels and small wave height with high water levels. These ten conditions will allow approximation of the worst-case overtopping discharge for each return period.

The corresponding extreme wave and water levels collected from node 919 are shown in Table 4-2 along with the corresponding wave periods which were calculated using the equation.

Table 4-2 SoN offshore extreme wave, water levels (updated to 2017), and peak wave periods

Event return period	Parameter	Design combination									
		1	2	3	4	5	6	7	8	9	10
20	Wave height (Hs)	4.00	3.95	3.74	3.63	3.53	3.05	2.45	1.80	1.14	0.53
	Water level (SWL)	0.05	0.83	1.35	2.00	2.72	2.79	2.93	3.02	3.07	3.11
	Wave period (Tp)	8.53	8.47	8.24	8.12	8.01	7.44	6.67	5.71	4.55	3.10
50	Wave height (Hs)	4.27	4.30	3.96	3.82	3.71	3.35	2.69	2.03	1.26	0.54
	Water level (SWL)	0.05	0.83	1.29	1.94	2.59	2.89	2.99	3.11	3.15	3.18
	Wave period (Tp)	8.80	8.84	8.48	8.33	8.21	7.80	6.99	6.08	4.78	3.14
100	Wave height (Hs)	4.57	4.58	4.15	3.94	3.83	3.35	2.69	2.03	1.26	0.54
	Water level (SWL)	0.05	0.90	1.29	1.90	2.65	2.94	3.07	3.19	3.24	3.26
	Wave period (Tp)	9.11	9.12	8.68	8.46	8.35	7.80	6.99	6.08	4.78	3.14
200	Wave height (Hs)	4.85	4.90	4.42	4.07	3.99	3.52	2.93	2.21	1.44	0.60
	Water level (SWL)	0.05	0.90	1.29	1.81	2.59	2.99	3.12	3.27	3.32	3.34
	Wave period (Tp)	9.39	9.44	8.96	8.60	8.52	8.00	7.29	6.34	5.11	3.31
500	Wave height (Hs)	4.95	5.10	4.72	4.30	4.15	3.82	3.05	2.33	1.44	0.72
	Water level (SWL)	0.05	0.83	1.29	1.75	2.59	3.11	3.21	3.38	3.41	3.45
	Wave period (Tp)	9.48	9.63	9.26	8.84	8.69	8.33	7.44	6.51	5.11	3.62
1000	Wave height (Hs)	5.01	5.20	4.96	4.48	4.37	3.94	3.17	2.45	1.68	0.84
	Water level (SWL)	0.05	0.77	1.31	1.76	2.65	3.14	3.27	3.45	3.50	3.51
	Wave period (Tp)	9.54	9.72	9.49	9.02	8.91	8.46	7.59	6.67	5.52	3.91

4.2.3 Climate Change Allowance

Account of increasing sea levels as a consequence of climate change is critical for establishing the future flood/erosion risk which the study area might experience. Projections for sea level rise in the study area were derived from the UKCP09 data (UKCP09, 2017) for the Torcross area. Following the latest climate change guidance (Environment Agency, 2016c), the Medium scenario 95%ile data was used to establish sea level rise allowances.

As the base date of the wave data in the SoN water level data was 2014. A sea level rise allowance of 0.017m was added to this original water level data to generate Year 0 data for 2017. Additional allowances for sea level rise were also established to estimate the water level for future dates (Year 50 and Year 100), as indicated in Table 4-3.

Table 4-3 Sea Level Rise allowance for Torcross

Year	Sea Level Rise (m)
2014 – Base date	0.000
2017 – Year 0	0.017
2067 – Year 50	0.325
2117 – Year 100	0.770

4.2.4 Nearshore Transformation

To determine the nearshore wave conditions required for overtopping design, the design conditions were transformed from the offshore SoN data point to the toe of the flood defences. Wave transformations were completed in accordance with the methodology described by Goda (2000). This method assumed the nearshore beach contours are straight, and parallel to the flood defences. It is considered that the study area at Slapton Sands meets these conditions.

4.2.5 Defence Geometry

For each overtopping location, the geometry of the defence was analysed with consideration of the beach profiles between 2007 and 2016 (CCO, 2017) and the information collected from the condition assessment. This data was simplified to create a profile corresponding with a low beach level, for use in overtopping calculations per EurOtop II (Environment Agency, 2016b). A low beach level was chosen to assess the conditions expected towards the end of a storm, or when beach volumes are generally depleted. Figure 4.5 shows the simplification of beach profile 6b01320 to indicate a typical outcome of this simplification process.

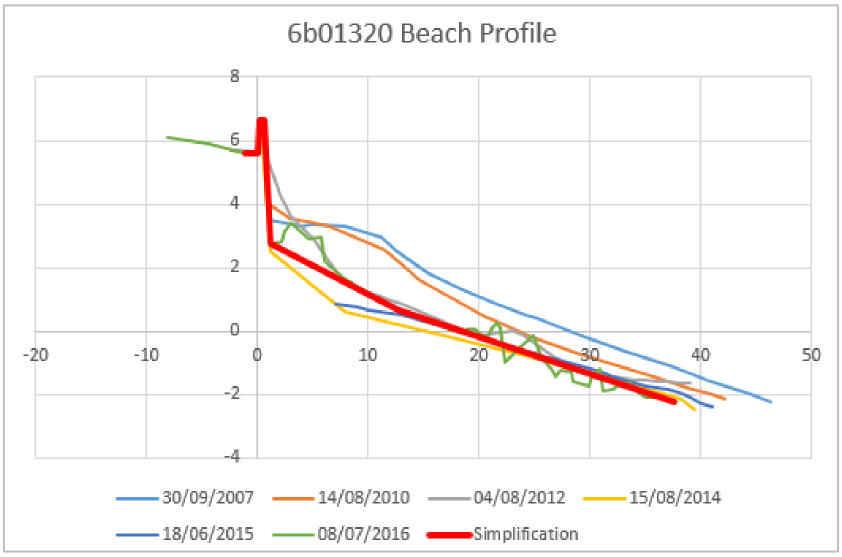


Figure 4.5 Example of profile simplification (6b01320)

4.3 Results

The following section outlines the results of the worst-case overtopping and run-up analyses for the joint probability combinations at each return period and defence section considered. All incident significant wave heights which created the largest wave overtopping within the analysis were between 3.27m and 5.24m, indicating limits for public and vehicle safety of between 0.3 and 1.0 l/s/m. The shading of the values in the tables below indicates whether there is a risk to pedestrians or vehicles based on the limits stated in Figure 4.2 and Figure 4.3. The green shaded boxes indicate overtopping values which do not present a hazard to pedestrians or vehicles, yellow shaded boxes indicate values which present a hazard to pedestrians, and orange shaded boxes indicate values which present a hazard to both people and vehicles.

4.3.1 Present Day

Table 4-4 indicates overtopping rates for the present-day scenario and Table 4-5 indicates whether wave run-up exceeds the crest level of the beach areas north of Torcross.

Table 4-4 Present day - Overtopping discharge (l/s/m). Green - shows no risk to public/vehicles; Yellow - shows a risk to the public; and Orange - shows a risk to both the public and vehicles

Profile	Extreme Water Level Return Period					
	20	50	100	200	500	1000
6b01320	0.00	0.00	0.00	0.00	0.00	4.18
6b01316	0.66	0.88	1.27	1.86	2.54	30.93
6b01314	0.19	0.28	0.42	0.59	0.86	20.16
6b01313	0.80	1.04	1.63	2.16	51.17	63.86
6b01311	0.10	0.15	0.25	0.36	0.57	1.12
6b01306	36.98	40.51	55.33	64.07	112.08	130.69
6b01278	8.20	9.71	15.48	19.48	40.74	54.32
6b01273	7.52	9.17	14.25	18.10	34.06	45.47
6b01258	5.11	6.68	10.46	13.66	27.18	37.37
6b01245	0.20	0.27	0.47	0.63	1.38	2.08

Table 4-5 Present Day Wave Run-up - Yes shows when the wave run up exceeded the defence level

Profile	Extreme Water Level Return Period					
	20	50	100	200	500	1000	Overflow?
6b01306	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01278	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01273	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01258	Yes	Yes	Yes	Yes	Yes	Yes	Yes

At present day (Table 4-4), the most significant area of concern for wave overtopping is in the area north of Torcross between the slipway and the northern end of the beach. This area includes profiles (6b01306, 6b01278, 6b01273 and 6b01258), describing the flood defence provided primarily by the gravel beach flood.

The overtopping analysis indicates relatively large overtopping rates, creating a risk to pedestrians and vehicles for all return periods examined. The high overtopping rates are suggestive that pedestrians and vehicles will be at risk during more frequent extreme events, although data was not available to test this assumption. The high overtopping rates generated by the tested conditions also suggest a high likelihood that the A379 would be closed due to debris being transported to the road, or damage to the roadway. This view is further supported by the results of the wave run-up assessment, which indicated the 2% run-up limit will exceed the beach/structure crest level for the worst case of all return periods examined. This indicates that the beach might roll back and flatten, transporting sediment on to the road and affecting access.

The more substantial flood defences at Torcross and the wide shallow beach at Strete gate might be expected to provide an elevated standard of protection. This assumption was supported by the analysis, with the lowest risk from overtopping occurring at Torcross (including profiles 6b01320, 6b01316, 6b01314, 6b01313 and 6b01311) and at the northern limit of the study area (profile 6b01245). Despite the lower overtopping rates in these sections, results indicate a risk to pedestrians during 5% AEP events (1 in 20-years). The exceptions being profile 6b01320, which shows minimal overtopping levels until 0.1% AEP (1 in 1000-year) and profiles 6b01316, 6b01313 and 6b01245 which show no risks until 1% and 0.05% AEP (1 in 100-year or 1 in 200-year respectively). It should be noted that there is a risk to vehicles at a 0.1% AEP event (1 in 1,000-year) across all profiles except 6b01311 and 6b01245 which indicated wave overtopping would not exceed vehicle safety or damage criteria even under the 0.1% AEP events.

4.3.2 Year 50

Table 4-6 indicates overtopping rates for the Year 50 scenario and Table 4-7 indicates whether wave run-up exceeds the crest level of the beach areas north of Torcross.

Table 4-6 2067 - Overtopping discharge (l/s/m). Green - shows no risk to public/vehicles; Yellow - shows a risk to the public; and Orange - shows a risk to both the public and vehicles

Profile	Extreme Water Level Return Period					
	20	50	100	200	500	1000
6b01320	0.00	0.00	0.00	0.00	0.00	5.90
6b01316	1.01	1.30	1.85	2.43	3.25	38.88
6b01314	0.30	0.42	0.63	0.85	1.22	25.82
6b01313	1.43	1.83	2.77	3.61	86.03	104.85
6b01311	0.18	0.27	0.45	0.64	0.98	1.85
6b01306	63.49	67.58	89.13	100.58	193.24	224.72
6b01278	18.29	20.36	30.73	36.88	75.78	96.65
6b01273	15.37	17.69	26.38	32.12	60.82	78.12
6b01258	11.23	13.67	20.55	25.58	51.39	67.54
6b01245	0.47	0.60	1.02	1.35	2.90	4.20

Table 4-7 2067 Wave Run-up - Yes shows when the wave run up exceeded the defence level

Profile	Extreme Water Level Return Period						
	20	50	100	200	500	1000	Overflow?
6b01306	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01278	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01273	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01258	Yes	Yes	Yes	Yes	Yes	Yes	Yes

The wave overtopping discharge values for 2067 are substantially higher than the equivalent values for the present-day scenario, with most frontages experiencing an approximate doubling in discharge volumes. Despite this, the number of events in which the overtopping rate exceeded the threshold for pedestrians and vehicle safety were broadly similar to the present-day simulations. The location where a change in the risk level had noticeably changed was at profile 6b01311, which indicated exceedance of the pedestrian safety limit to a 1% AEP (1 in 100-year) event, and profile 6b01245 which indicated exceedance of the pedestrian safety limit to at least a 5% AEP (1 in 20-year) event. The high overtopping rates in the beach area are likely to result in damage to the areas adjacent to the road, causing undermining and washout of fill material.

Examination of wave run-up indicated that the 2% run-up limit exceeded the beach/structure crest level for the worst case of all return periods examined. The level of run-up was more significant than the present-day scenario, introducing additional material to the highway, and increasing the risk of damage to the road surface and adjacent areas.

4.3.3 Year 100

Table 4-8 indicates overtopping rates for the Year 100 scenario and Table 4-9 indicates whether wave run-up exceeds the crest level of the beach areas north of Torcross.

Table 4-8 2117- Overtopping discharge (l/s/m). Green - shows no risk to public/vehicles; Yellow - shows a risk to the public; and Orange - shows a risk to both the public and vehicles

Profile	Extreme Water Level Return Period					
	20	50	100	200	500	1000
6b01320	0.00	0.00	0.00	0.00	0.01	9.68
6b01316	1.80	2.26	3.12	3.92	5.17	53.73
6b01314	0.57	0.76	1.10	1.45	2.02	36.80
6b01313	2.91	3.84	5.56	7.08	171.54	203.88
6b01311	0.40	0.59	0.95	1.35	2.03	3.62
6b01306	128.71	132.65	167.11	188.23	322.07	370.41
6b01278	52.57	55.15	76.25	92.34	167.99	202.48
6b01273	40.80	43.92	61.34	70.74	130.63	159.47
6b01258	33.14	36.76	51.87	60.62	118.12	146.29
6b01245	1.54	1.83	2.95	3.73	8.23	11.36

Table 4-9 2117 Wave Run-up - Yes shows when the wave run up exceeded the defence level

Profile	Extreme Water Level Return Period						
	20	50	100	200	500	1000	Overflow?
6b01306	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01278	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01273	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6b01258	Yes	Yes	Yes	Yes	Yes	Yes	Yes

The wave overtopping discharge values for 2117 are substantially higher than the equivalent values for the present-day scenario or the 2067 scenario, with most frontages experiencing an approximate doubling or trebling in discharge volumes from the 2067 scenario. Several frontages indicated a change in frequency of exceedance of the pedestrian and vehicle safety thresholds. All areas except the most southerly section of Torcross (profile 6b01320) exceeded the pedestrian safety limit in all tested events, with high values for even the most frequent return periods tested. It is likely that pedestrian safety would be exceeded by more frequent events than those tested. Exceedance of vehicle safety threshold was more frequent with profile 6b01316 exceeded during a 0.2% AEP (1 in 500-year) event; profile 6b01313 exceeded during a 1% AEP (1 in 100-year) event; and profile 6b01245 exceeded during a 0.2% AEP (1 in 500-year) event.

Examination of wave run-up indicated that the 2% run-up limit exceeds the beach/structure crest level for the worst case of all return periods examined. The level of run-up is more significant than the present-day scenario and 2067 scenario, introducing additional material to the highway, and increasing the risk of damage to the road surface and adjacent areas.

4.4 Comparison with JBA Work

The overtopping analysis completed by JBA for Torcross was part of a wider project which estimated wave overtopping around the South West coast, and examined two profiles. The analysis considered a single profile at each location, with simplification of defence/beach geometry based on the 2015 survey that corresponded to a low beach level.

This section provides a comparison between the analysis completed for the BMP (described above) and that undertaken by JBA.

4.4.1 The Key Differences

- **Different beach profile:** The overlap in profiles examined by CH2M and JBA is relatively good. Both organisations examined overtopping at profile 6b01313. CH2M examined profile 6b01320, while JBA examined the adjacent profile, 6b01319.
- **Different return periods:** There was less consistency in the return periods examined by both organisations, with the return periods identified by JBA having relatively small overlap with the return periods provided in the SoN dataset used by CH2M. Events examined by both organisations included return periods of 1:20-year, 1:200-year and 1:1,00-year.
- **Different wave periods and directions:** The data adopted by JBA includes modelled data close to the shoreline, which indicates waves with large periods approaching the shore from the southeast. The analysis in this report calculated smaller wave periods, and assumed waves approach from the east, which represents a worst-case scenario and is broadly consistent with a significant component of the recorded data from the Slapton Directional Wave Rider Buoy.
- **Different analytical approaches:** The analysis by JBA capitalised on the processing efficiencies of the neural network tool to examine overtopping for many conditions and locations. However, it is unclear whether the analysis followed EurOtop guidance from 2007 and 2016, and which confidence level was selected.

4.4.2 Implications of the Key Differences

- Overtopping discharge rate is highly dependent on wave period, therefore differences in outputs between the two analyses would be expected due to differences in this input parameter;
- It is noteworthy that the defence changes between profile 6b01320 and 6b01319, therefore the difference in results between these two profiles may be attributed to the different cross-shore profile of the defences; and
- The comparison indicates that the results produced by CH2M were generally lower than the equivalent values produced by JBA. The exception to this is at very high return periods for profile 6b01313, where CH2M indicate a substantial increase in discharge rate, exceeding the rate estimated by JBA. Overall the differences in results produced in this report and by JBA are not unexpected, due to the sensitivity of the analysis to changes in some parameters.

SECTION 5

Conclusions

5.1 Defence Condition

Generally, the condition of the existing defence assets is Good, however there are a few sections in worse condition. These sections will require remedial works in the shorter term to avoid further failure and property/asset damage during storm events. These areas are as follows:

1. Frontage 5 has a 50m concrete seawall defence which has been graded as 4 (Poor) with large areas of significant cracking, spalling and minor displacement. If this defence is not repaired in the short term it is estimated to completely fail within the next 15 years;
2. 'Armourflex' blockwork at Frontage 7 was assessed as condition grade 5 (Very Poor), which indicates the defence has already reached complete performance failure. The nylon cables holding the blocks together were broken in several places, and the underlying geotextile was damaged. Failure to rectify this defence will likely result in further damage to the amenity area behind the beach, and outflanking of the car park.
3. The middle carpark embankment (Section 8) was also assessed as condition grade 5 (Very Poor). The tarmac of the parking area and underlying substrate had been undermined and eroded resulting in loss of parking capacity. Failure to rectify this defence will likely result in ongoing damage to the car park, and further reduction in parking capacity.

5.2 Overtopping

5.2.1 Data and Methodology Discussion

The analysis has several limitations which might affect the outcome of the analysis. The SoN data used in the analysis is presently being re-examined to confirm its accuracy for analytical purposes. If there are significant changes to the updated SoN data, the results of this analysis will need to be revisited. It should also be noted that the SoN did not have data available for more regular events with smaller return periods, which would prove useful in assessing whether work is needed in the short-term to address flood risk during more frequently occurring storm events.

The wave and water level data from SoN also did not include a description of peak wave period angle of wave approach in the stated joint probability conditions. The approach outlined by Boccotti (2000) was used to estimate the peak wave period, although the applicability of the relationship to the SoN data point is unclear. If longer duration period waves can approach the shore, normal to the defences, the outputs of the wave overtopping analysis will under-predict the overtopping discharge rates. This is considered unlikely due to the somewhat limited fetch length of shore-normal wave approach. Longer duration waves may have been recorded at the site, however these are likely to be associated with waves that are only partly refracted around Start Point. Such waves are likely to approach the coast at oblique wave angles, and would therefore experience more energy dissipation during wave breaking and run-up. The assumption that all waves approach flood defences at an angle normal to the flood defences has contributed to conservative estimates for wave overtopping.

The simplification of the beach profiles is a further area of potential conservatism in the analysis. Assuming a simplified post-storm profile ensures beaches with lower foreshores and steeper beaches are included in the analysis. This resulted in lower energy dissipation than higher beaches with a shallower foreshore, increasing the overall discharge rate. This approach was adopted to provide an indication of the overtopping discharge toward the end of a storm event, however it is possible that the beach might not reach this profile state during a storm, leading to an over-estimation of overtopping discharge.

5.2.2 Results Discussion

The results indicate that the area at most significant risk of flooding by overtopping of coastal defences in the present day is the area between the Torcross slipway and the northern beach adjacent to the Higher Ley. This length of coast is characterised by a gravel beach acting as the primary flood defence, with only small structures at the crest of the beach area. The limit for public safety and vehicle safety is exceeded in this area for all the present-day and future conditions examined. The large overtopping rates are also likely to exceed the design structural stability limits throughout this stretch (assumed to be 0.1 l/s/m based on Figure 4.3, as much of the area behind the defence is grassed bank).

The flood defences at Torcross generally provide a much higher standard of protection than the adjacent beach area further north. Pedestrian safety behind the Torcross flood defences is variable, but falls below the 5% AEP (1 in 20-year) event in two places. The vehicle safety threshold (5 l/s/m) is broadly not exceeded until the 0.2% AEP (1 in 500-year) event, with one exception at 6b01313 (the section of steel sheet pile at the crest of the shingle beach). The threshold for vehicle safety is the same as the likely damage threshold for the flood defences at Torcross (5 l/s/m assuming Rubble mound breakwaters with rear side designed for wave overtopping in Figure 4.3). As such, the most likely area of damage during future extreme events will be at the stretch of steel sheet piling.

Wave overtopping discharge rates significantly increase at future dates. For the return periods examined, the results at Year 50 indicate little change in the risk to pedestrians or vehicles behind the flood defences. It is likely that pedestrian safety may be exceeded during lower return period events. The increased discharge rates along the main beach section will lead to more significant damage to the upper beach and road area.

For the return periods examined, the results at Year 100 indicates a marked change in the risk to pedestrians and vehicles behind the defence. The most noted area of changes are at Torcross, where the flood defences fall below the 0.5% AEP (1 in 200-year) design level, and at Strete Gate, where damage, overwash, and potential breach of the beach might be expected for events with 0.2% AEP (1 in 500-year).

It is recommended that the overtopping results provided in this baseline report are sense checked against the JBA overtopping modelling results (being undertaken as part of the Slapton Sands BMP for the end of August 2017) once completed.

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