SEATON BEACH MANAGEMENT PLAN

Defences Baseline

Prepared for

East Devon District Council

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Introduction

1.1 Background and Study Area

This report has been prepared for the East Devon District Council as part of the Seaton Beach Management Plan (BMP), and provides a baseline assessment of the coastal defences located along the Seaton BMP frontage between Seaton Hole and the east bank of the River Axe at the river mouth (Figure 1-1).

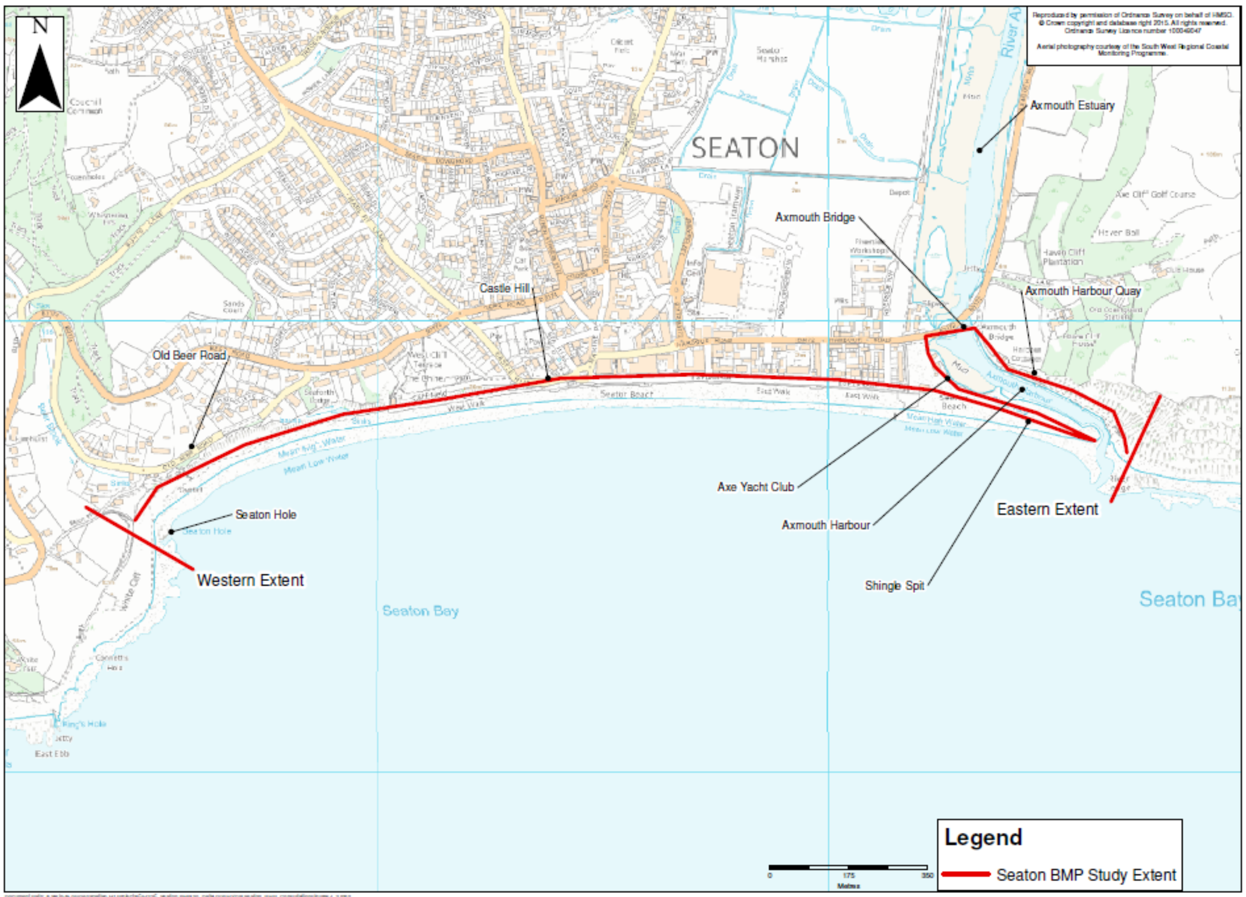


Figure 1-1 Seaton BMP study area
Map showing the extent of the BMP study area

1.2 The Basis of the 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 Seaton 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.
- Reporting the wave overtopping rates for a range of extreme events, and calculation of the standard of protection provided by the existing coastal defences (Section 4), based on outputs of previous analysis.
- A new assessment of the required changes to the cross-shore profile to avoid the reported problem of the beach rolling back onto the seawall under extreme events (Section 5).
- Conclusions and recommendations for further investigations (Section 6).

SECTION 2

Defence History

This Section provides an outline of the coastal defence history along the BMP frontage (Figure 2-1), from Seaton Hole to the east bank of the River Axe at the river mouth, in order to understand the previous approaches to flood and coastal erosion risk management (FCERM).

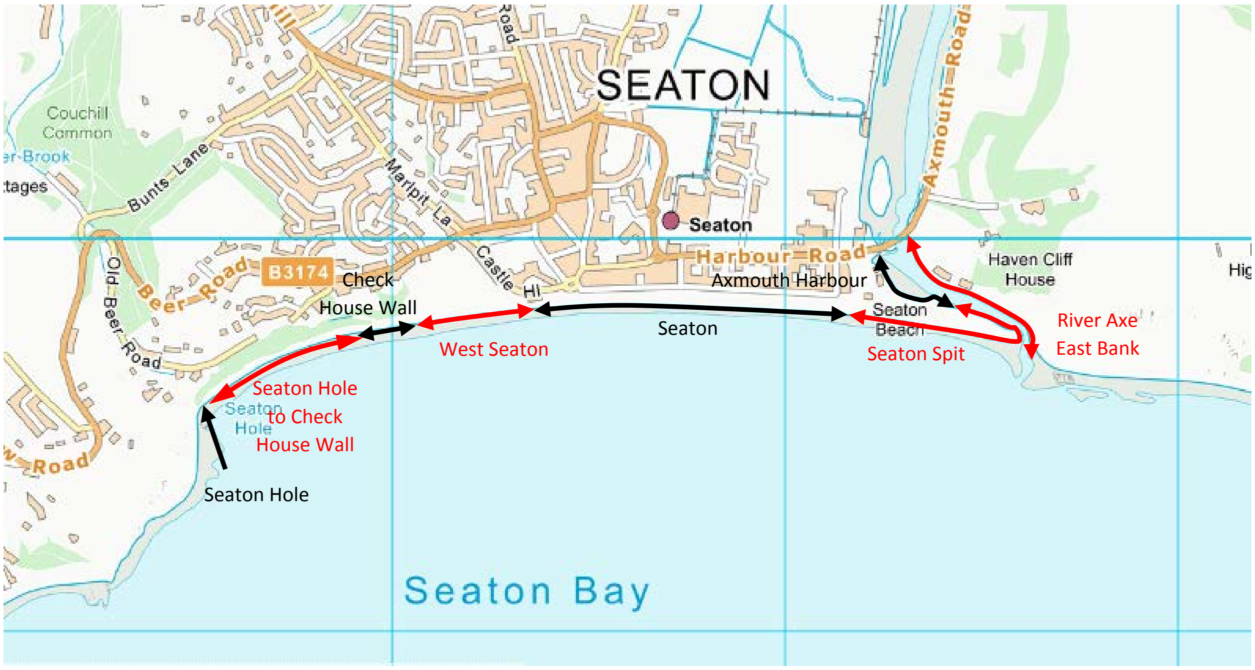


Figure 2-1 Defence history frontages

Map showing the extent of the frontages used to describe the defence history (Map copyright Ordnance Survey, 2017)

2.1 Seaton Hole to West Seaton

2.1.1 Seaton Hole

Seaton Hole is a located at the base of the cliffs below Old Beer Road to the west of Seaton. The location includes an outfall which has experienced ongoing issues of erosion, leading to undermining and outflanking. Recent changes at this location include:

- In the 1970s, scree material from the base of the White Cliffs was placed at Seaton Hole to prevent further erosion. This was later encased in concrete to prevent their displacement during wave attack. In 1994, an inspection recorded significant exposure of the structures toe (Posford Duvivier 1994).
- A major landslip occurred in the autumn of 2000. The incident resulted in a culvert outlet at the western end of the existing defences at Seaton Hole being obstructed by landslip debris (EDDC, 2000).
- In 2002/2003, wing walls and a head wall were constructed around the culvert outlet/outfall to protect the area from obstruction. The wing walls and head wall was constructed from gabion baskets filled with concrete sand bags (Figure 2-2).

- The gabion baskets protected the culvert outlet/outfall during the winter storms in 2004, however a significant volume of landslip debris was washed away along with the loss of many loosely placed concrete-filled sand bags. This allowed some erosion at the western edge of the structure (David Roche GeoConsulting Limited, 2003).
- In 2005, further storms resulted in the loss of additional concrete-filled sand bags, and works were undertaken to strengthen the wing walls (David Roche GeoConsulting Ltd, 2005). The work included the introduction of additional gabion baskets filled with concrete-filled sand bags in front of the western wing wall as well as extending the wing wall further west (Figure 2-3).

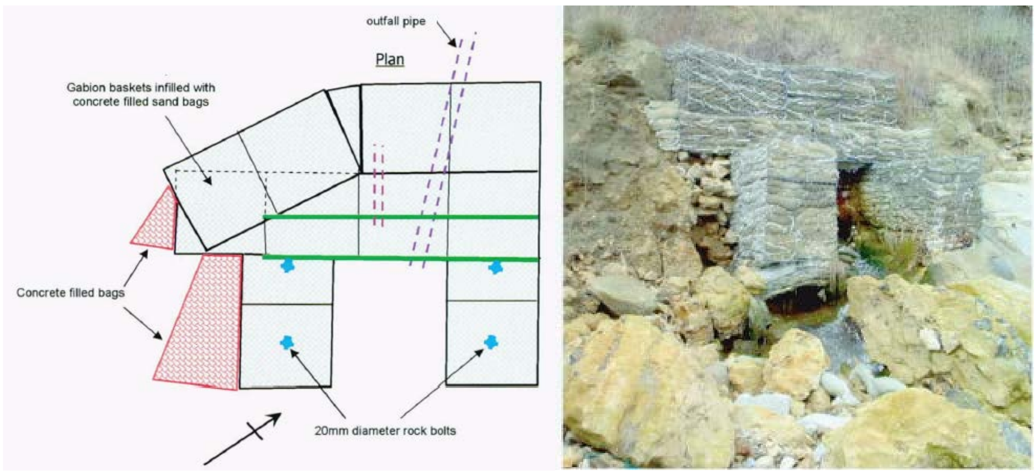


Figure 2-2 Outfall protection works with destroyed areas in 2004/2005 shown in red and in the adjacent photo.
Source: David Roche GeoConsulting Ltd (2005)

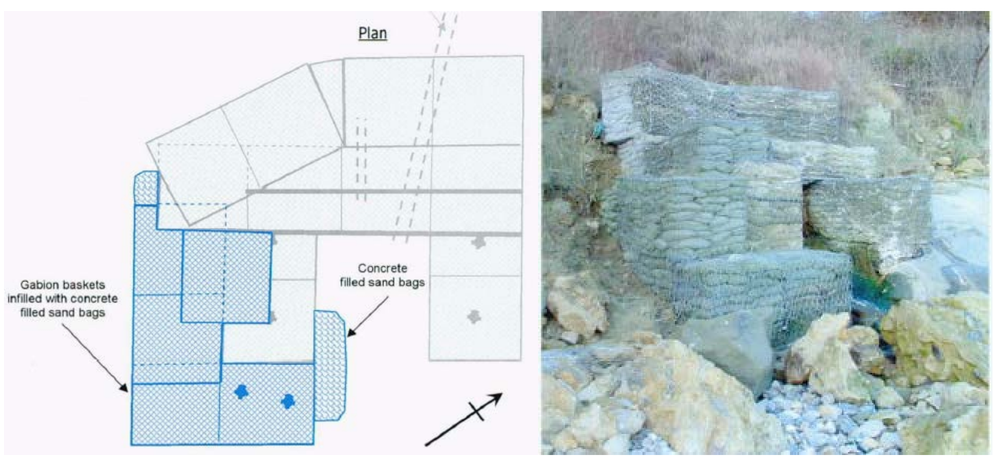


Figure 2-3 Outfall protection works constructed in 2005 shown in blue and completed in the adjacent photo
Source: David Roche GeoConsulting Ltd (2005)

2.1.2 Seaton Hole to Check House Wall

The coast between Seaton Hole and Seaton is backed by high cliffs, with most of the defence measures provided by protection at the toe of the cliffs. Details of the current defences along this frontage are shown in Figure 2-4 and Figure 2-5, which were published by EDDC (no date) as part of a publicity document for the scheme. The overall defence history for this frontage is as follows:

- In 1989/90 the beach between Seaton Hole and Seaton was severely drawn down, exposing the toe of the cliff and underlying rock foreshore to wave action. Erosion of the cliff put the Old Beer Road and adjacent properties under increasing threat of erosion.

- Sometime prior to 1994, a rock revetment, approximately 65m in length, was constructed to stop further erosion at the cliffs east of the encased concrete defence. The rock revetment consisted of 3-5 tonnes of Mendip limestone which was placed along the toe of the cliff with an average crest height of 6.2m OD and front slope of 1:1.5 (Posford Duvivier, 1994).
- In 1998 a £550,000 scheme to construct a rock revetment at the base of the cliffs was designed by Posford Duvivier Consulting Engineers and in 1998/1989 constructed by Costain Civil Engineering Ltd. The revetment was intended to absorb wave energy and reduce the rate of erosion along this stretch of coast. The revetment extended approximately 410m east from Seaton Hole to the older concrete wall to the west of the main Seaton flood defences (Figure 2-4), and was formed of 5 to 8 tonne limestone rock for the primary armour.

  The structure adopted two cross-sectional profiles (outlined in Figure 2-5) that were applied to different parts of the frontage depending on the perceived erosion risk. The single-sided revetment was adopted in sections where cliff recession was stable, while the double-sided revetment was used in more unstable sections.

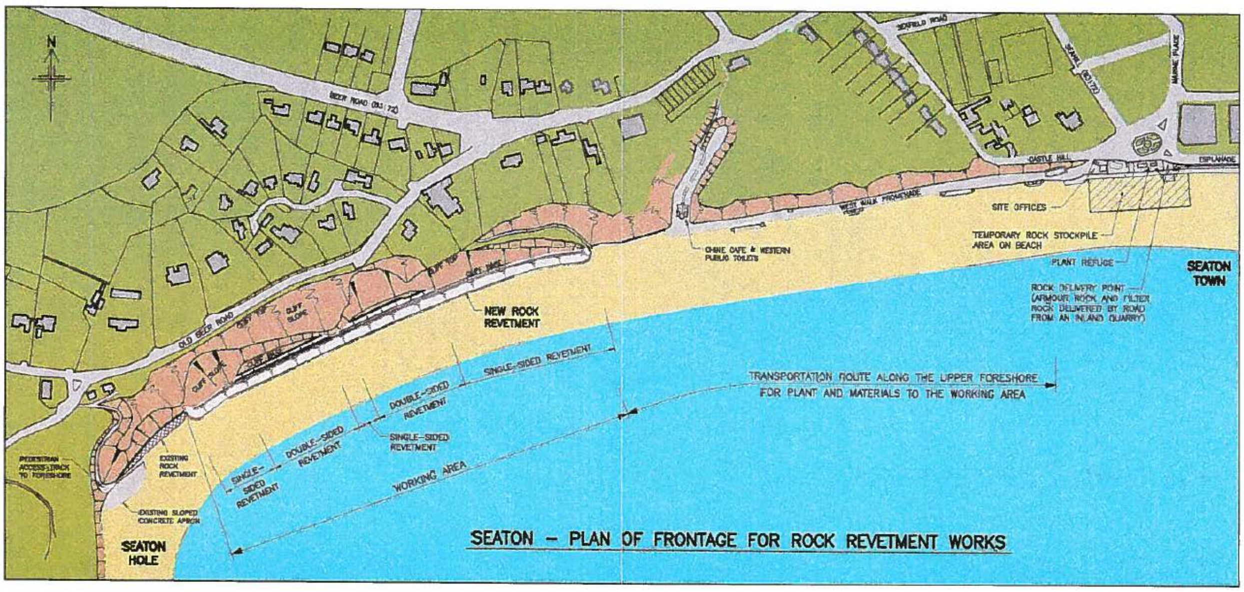


Figure 2-4 Rock revetment works between Seaton Hole and Seaton
Source: (EDDC, no date)

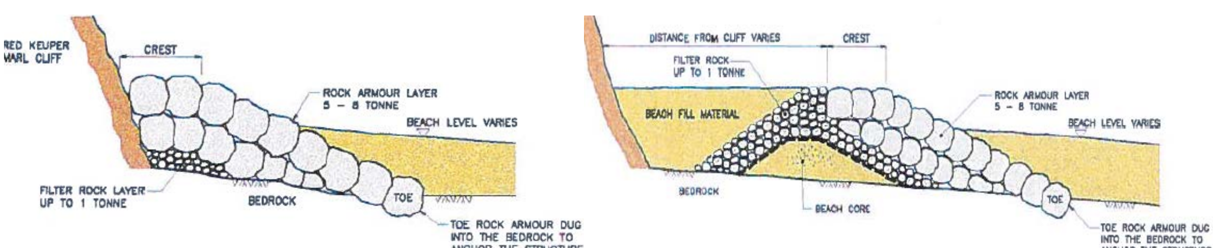


Figure 2-5 Typical section of single-sided revetment (left) and Typical section of double-sided revetment (right)
Source: EDDC (no date)

2.1.3 Western End of Check House Wall

The western end of the Check House Wall consists of gabion baskets and has been badly damaged in past and present storm events. Recent changes to the defences in this area have included:

- In July 2003 David Roche GeoConsulting inspected the area and recommended the additional beach/cliff protection works (EDDC, 2005). Details of the works are outlined in Figure 2-6.
- The beach/cliff protection works included the introduction of gabion baskets infilled with concrete-filled sand bags, and construction was completed by Celtic Rock Services during October 2003 (David Roche GeoConsulting Limited, 2003), as shown in Figure 2-7.
- In 2005, part of wider renovation work in the area by Bridge Civil Engineering Ltd included the addition of further stone-filled gabions (EDDC, 2005).
- In March 2015, an inspection of the Check House Wall and the western cliff area was completed. The inspection revealed that the stone filled gabions had suffered significant damage with more than half destroyed and the remaining gabions severely distorted and in danger of collapse (David Roche GeoConsulting Limited, 2015). Figure 2-8 shows the damage identified in 2015 compared with post-construction in 2005. Recommendations were made for remedial works with an indicative cost estimate between £20,000 and £30,000 (excluding VAT).

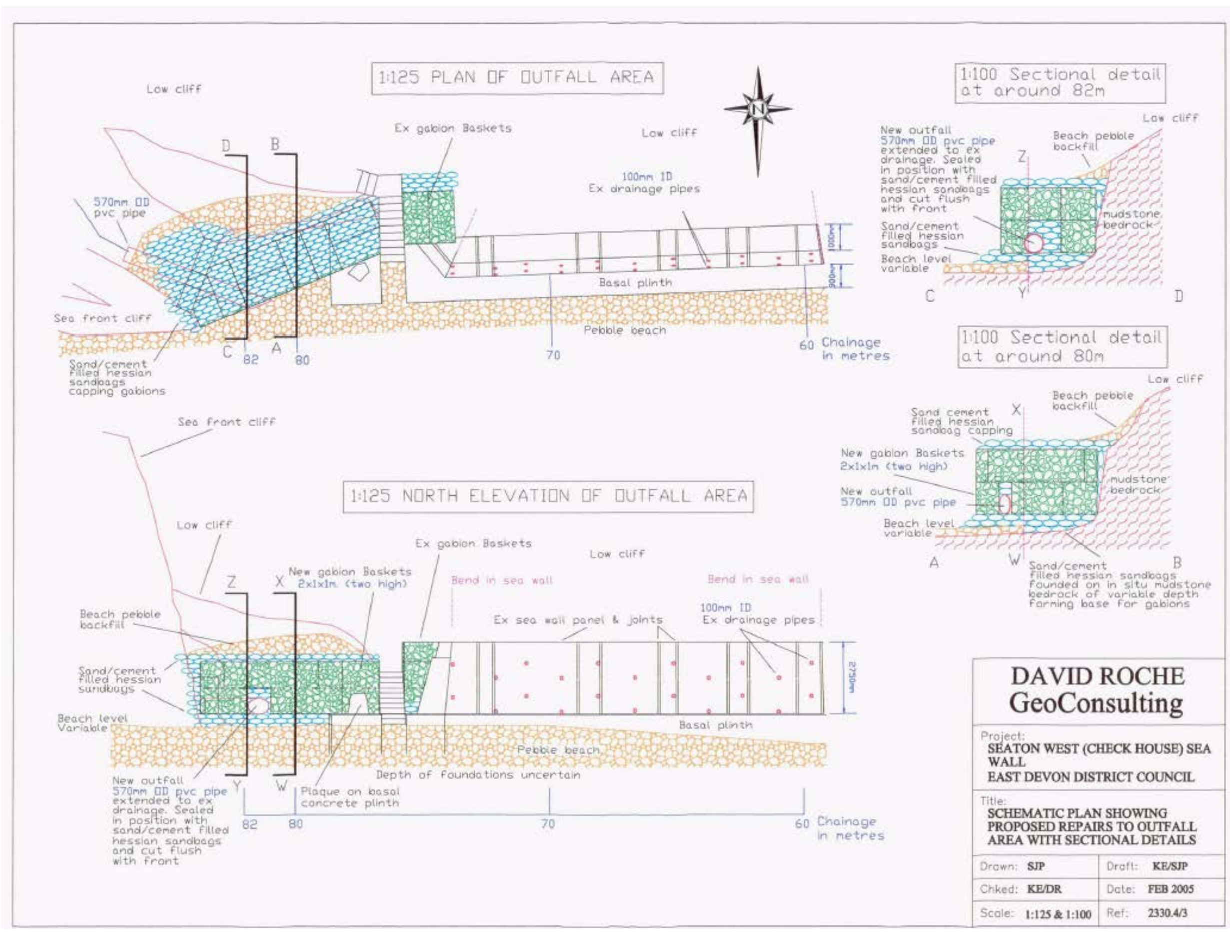


Figure 2-6 Schematic plan showing proposed repairs to the outfall area with sectional details
Source: David Roche GeoConsulting Limited (2005)

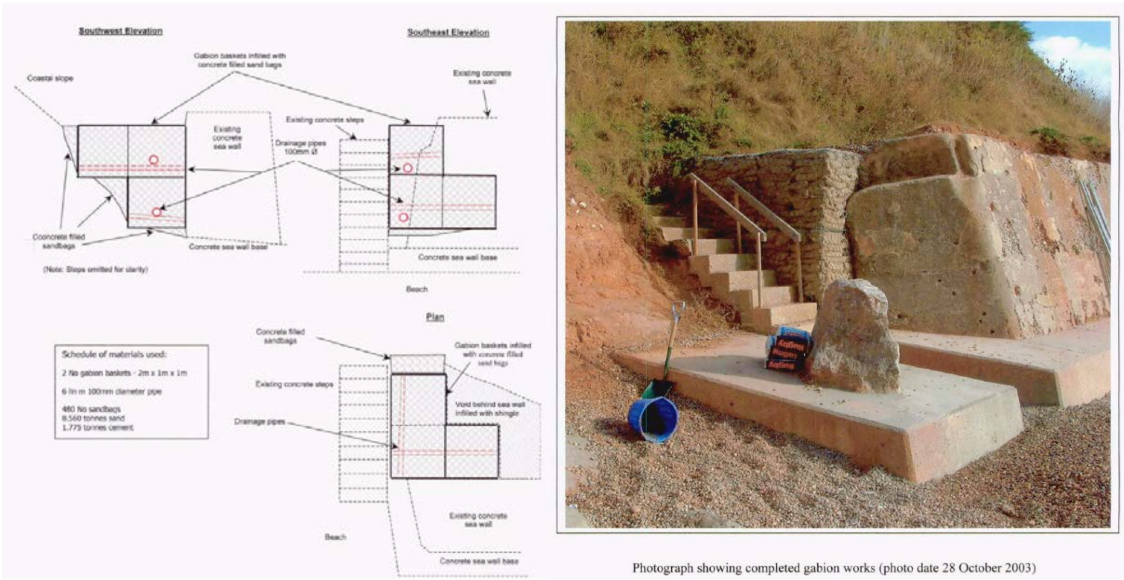


Figure 2-7 Side elevation plan of the emergency gabion works west of Check House Wall
Source: David Roche GeoConsulting (2003)

Figure 2-8 Construction photograph (2005) alongside the inspection photograph (2015)
Source: David Roche GeoConsulting (2015)

2.1.4 Check House Wall

The Check House Wall is located between the previously discussed rock revetment and the flood defences protecting the promenade to the west of Seaton. The wall consists of reinforced concrete with drainage holes and a recent sprayed concrete cover. The overall defence history of the wall is as follows:

- A 60m long concrete gravity wall was constructed to provide erosion protection for the cliff between the rock revetment and the flood defences protecting the promenade to the west of Seaton. It is unknown when the wall was originally constructed, however in a survey in the early 1990s recorded the wall to be in very poor condition with its foundations exposed (Posford Duvivier, 1994).
- In 1995, serious structural problems were identified at the concrete wall, and was highlighted as likely to fail at any time. In 1995/1996, EDDC carried out underpinning protection works to address the concerns, with work estimated to cost £35,000 (Posford Duvivier, 1995a).
- In July 2003 David Roche GeoConsulting inspected the wall and reported its condition as poor, recommending the wall should be upgraded with a skin of fine concrete placed in-situ; and

- Renovation of the Check House Wall was completed by Bridge Civil Engineering Ltd in 2005. The renovation included the introduction of steel mesh reinforcement, the application of 150mm thick layer of sprayed concrete (gunite) to the existing wall (EDDC, 2005). Existing drainage channels through the structure were also extended (Figure 2-9).

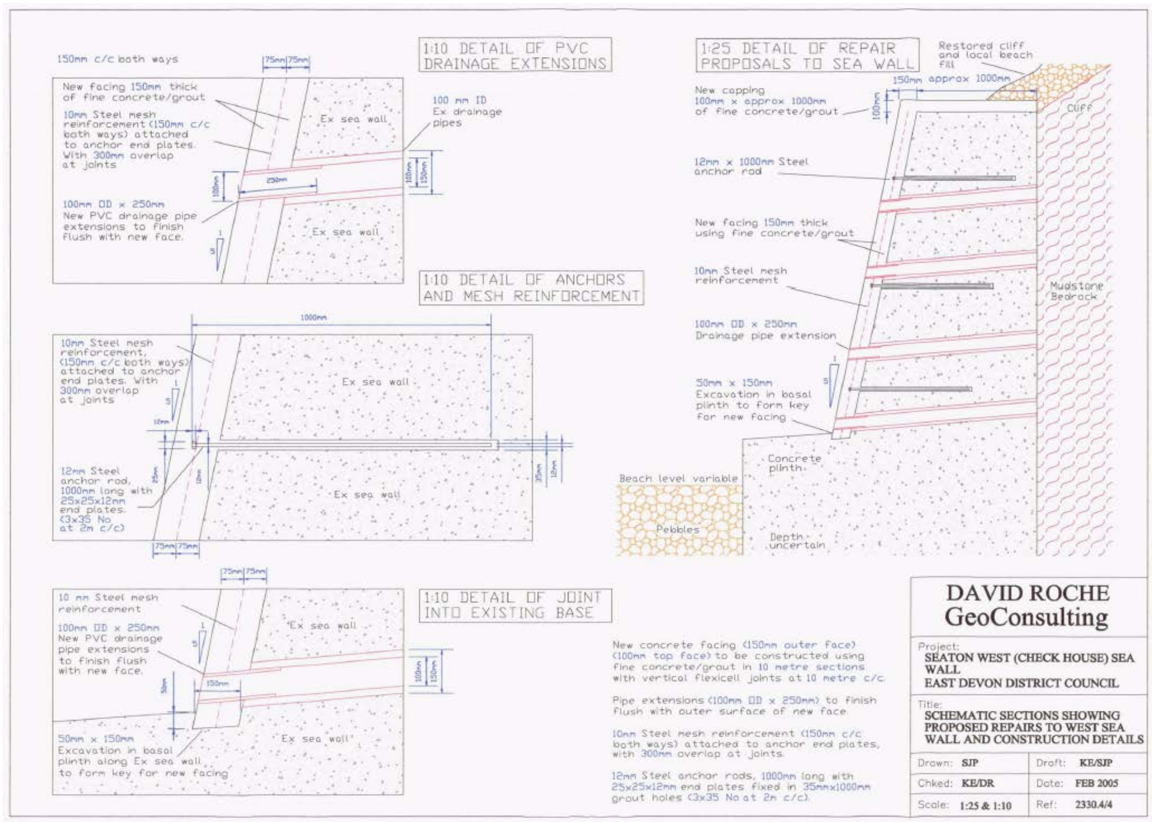


Figure 2-9 Schematic section showing proposed repairs to the west sea wall and construction details

Source: David Roche GeoConsulting (2005)

2.2 West Seaton

West Seaton includes a promenade protected by a near-vertical seawall, and includes integrated access steps and ramps. The historical change along the frontage outlined below:

- Prior to 1915 a promenade extended between Seaton and Seaton Hole, however it suffered significant damage in during storms in 1915 and was not replaced. (Posford Duvivier, 1994).

- Prior to the 1960s, the West Walk Promenade was reconstructed along the base of the cliffs, creating a link between Seaton and the café close to the Check House Wall (today the Hideaway Café) (Posford Duvivier, 1994). The West Walk Promenade was protected by a vertical concrete seawall.

- Remedial measures were undertaken in 1993 to protect the toe of the West Seaton promenade seawall due to concerns regarding undermining and collapse (Posford Duvivier, 1994).

- Posford Duvivier (1994) highlighted that the previous measures to protect the toe of the West Seaton promenade were insufficient and the seawall was in danger of collapse due to overturning or subsidence. During 1996/1997, improvements were made to this section of flood defence, protecting the promenade with the construction of a 370m long concrete seawall, dressed with stone blockwork (Figure 2-10) (Posford Duvivier, 1995b). This structure encased the older vertical concrete structure.

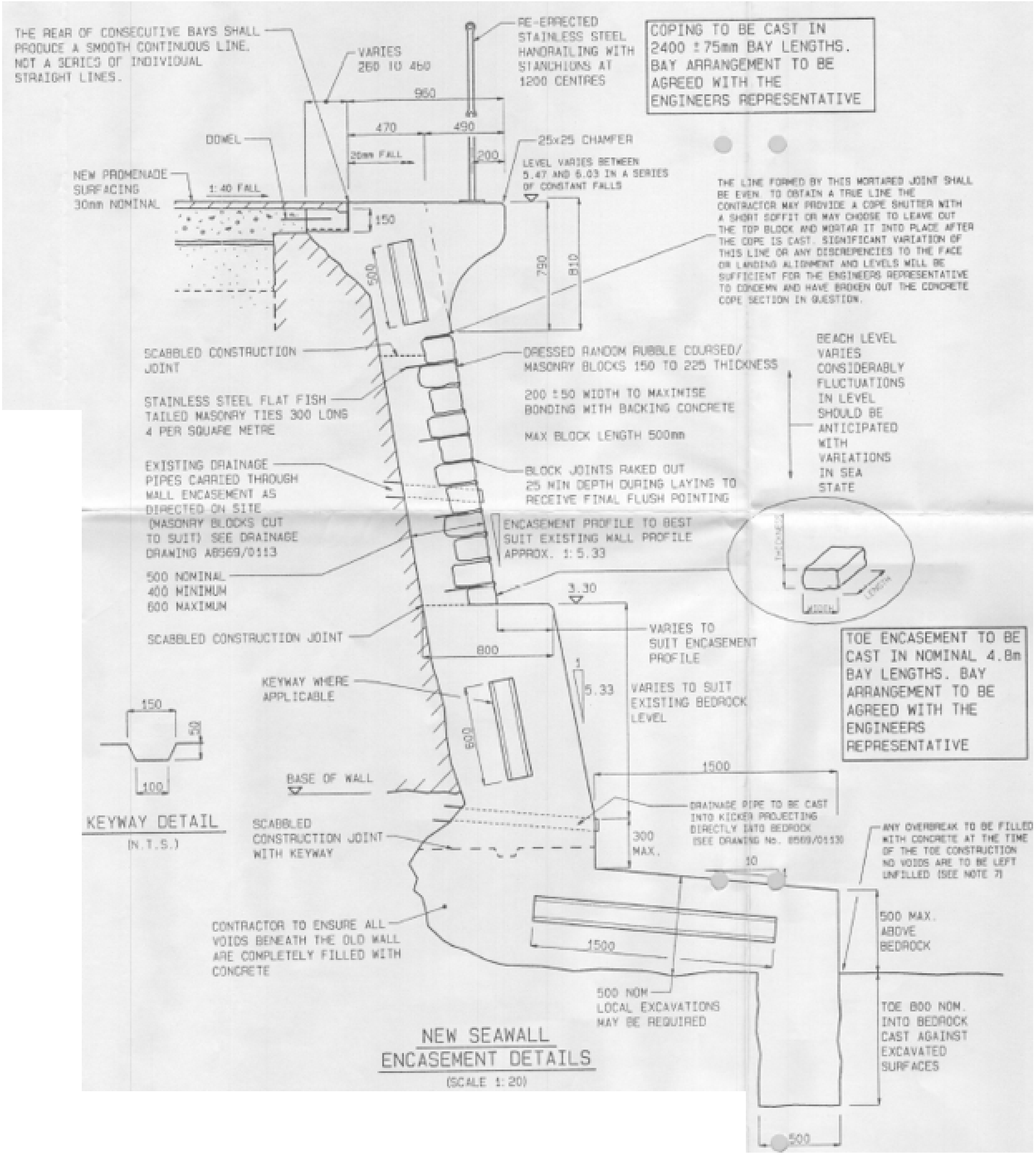


Figure 2-10 Revised seawall design for flood defence protecting the promenade to the west of Seaton

Source: Posford Duvivier (1995b)

2.3 Seaton

The main defence protecting Seaton Town include is a concrete seawall that runs adjacent to the Esplanade. The seawall is approximately 780m long, and includes integrated access steps and ramps, flood gates and the main slipway. The Environment Agency is currently considering modification to the main slipway gates to improve safety during operation. This would also improve the baseline condition of the assets.

The 780m long concrete seawall and wave return wall was constructed in 1980 by the South West Water Authority (now property of the Environment Agency) in response to severe flooding and damage following storms in 1979 (Posford Duvivier, 1994). The wall protects the Esplanade in Seaton between the main slipway and the eastern end of the esplanade, adjacent to the Axe Yacht Club clubhouse.

2.4 Seaton Spit

Seaton spit is largely undefended, with no defences constructed on its seaward face. However, development of the estuary inside of the spit into a harbour basin, starting in the 1970's, has resulted in significant modification due to the construction of defences around the harbour basin and the disposal of dredge material within trenches dug into the spit. This is discussed in more detail within Section 2.5. Storms have resulted in changes to the spit over the years, as described below:

- In 1979 a storm severely damaged the Axe Yacht Club House and the Spit was overtopped. After this event the channel and spit had to be reformed by the Council (CH2M, 2015a).
- In 2000 severe gales and excessive flood water caused erosion to the end of the spit, over widening the mouth and leading to significant erosion along the inner side of the spit.

2.5 Axmouth Harbour

Axmouth Harbour is protected from the substantial wave action by the Seaton Spit. The land adjacent to the western side of the channel is occupied by the Axe Yacht Club (AYC), which includes 105 moorings, and along the eastern side of the channel exists the eastern quay wall, where the local registered fishermen operate (discussed in detail in Section 2.6). The defence history of the area is described below:

- Axmouth Harbour was a small trading port until 1870, when a storm event caused damage to the harbour, which was left unrepaired and caused the harbour to become derelict.
- Trot moorings were established in 1967, and the harbour was regenerated in the 1970s through voluntary work and support from EDDC, who owned the harbour downstream of the highway bridges (Author unknown 2, 2000).
- Maintenance dredging of the channel was periodically undertaken to improve access to the harbour. In 1970, deposition of dredge spoil material was used to strengthen the shingle spit, increasing its width and height. Between the 1970s and 2015, the spit has reportedly been widened and raised two meters above the former shingle bank crest level (CH2M, 2015a).
- In 1977 part of the shingle beach inside the spit was excavated by the AYC (with support from the EDDC and approval from NRA) to create a mooring basin with a gabion bastion at its seaward end. A fence was built along the basin to protect the bank at high water (Author unknown 2, 2000).
- In 1988 Axmouth Harbour Management Company Ltd was formed. This organisation holds a lease from the EDDC for most of the land and defences in the harbour (Author unknown 2, 2000).
- In 1993 an investigation into dredging methods to prevent estuary blockage by silt was undertaken by A-W-Wakefield. This indicated that stabilised flow suction dredging using an in-line portable jet-

pump offered a viable solution for silt removal from the channel. This method was subsequently used for nine years, carried out by the AYC on behalf of the Axmouth Harbour Management Company and EDDC (Environment Agency, 2008).

- In 1995/6 secondary fencing was installed with rock armouring downstream of the harbour and bastion on the west bank of the River Axe (Author unknown 2, 2000).
- In 1996 the Axe Harbour Management Company were granted an exemption under the Waste Management Licensing Regulations 1994 by Devon County Council for the disposal of dredge spoil. Since 1996, dredging has continued annually, and Flood Defence Consent has been granted by the National Rivers Authority and the Environment Agency with no requirement for further consideration of a Waste Management License or exemption.
- In 2000, severe gales and excessive flood water caused erosion to the end of the spit, over-widening the mouth and leading to significant erosion along the inner side of the spit. This event also caused damage to the protective bastion for the mooring basin, undermining of the protective fencing within the basin, and erosion of the inner side of Seaton Spit (Axmouth Harbour Management Company Ltd, 2001).
- Between 2001 and 2003 the defences within the harbour basin were revised to address erosion issues, with the final structure involving reno mattresses and gabions (Axmouth Harbour Management Company Ltd, 2003). The defences are shown in Figure 2-11 and Figure 2-12.
- The conversion from trot moorings to a marina style pontoon finger berth was initiated in 2007 with three pontoons ordered from the Solent Marina of Southampton (AYC, no date).
- Between 2007 and 2009 pontoons and finger berths were installed (shown in Figure 2-13), increasing the capacity of the harbour (AYC, no date). No significant changes to the flood defences were made to accommodate the changes.
- From 2008, dredging from the harbour has been pumped into a reception trench excavated in the shingle spit above MHWS (CH2M, 2015a). An example of the trench that is dug when discharging dredged material is shown in Figure 2-14.
- In 2015 scour inspection by Royal Haskoning recommend the adoption of an Annual monitoring regime, and appropriate remedial works on ad hoc basis (Royal Haskoning, 2015 - Axmouth Harbour Entrance Scour Report).
- On the 25th June 2015, the Environment Agency set the following requirements for granting a D1 exemption for the disposal of silt dredged annually (Axmouth Harbour Management Company Ltd, 2015):
    - Strong flood risk assessment;
    - An investigation to show the dredging has minimal effect on flora and fauna; and
    - Change in method to a ‘swing shovel’.
- The Axe Harbour Management Company Ltd responded to these requests with the following comments (Axmouth Harbour Management Company Ltd, 2015):
    - They had positive results form a dye test in December 2013 stating that the suction dredging was highly efficient and caused minimal silt disturbance;
    - Statement from EA following a visit in 2013 stated that changes to the method statement could be made to advise the biodiversity team of the steps being taken to avoid damaging the flora and fauna. This included a map of the dredge area, notes

regarding the buried pipe and the fact that the flora and fauna was a direct result of silt being combined with shingle; and

- o Lastly comments were made on how impractical 'swing shovel' dredging would be in the harbour.

Figure 2-11 Photos of the protective fencing undermined and the construction of the gabion groyne
Source: Axmouth Harbour Management (2003)

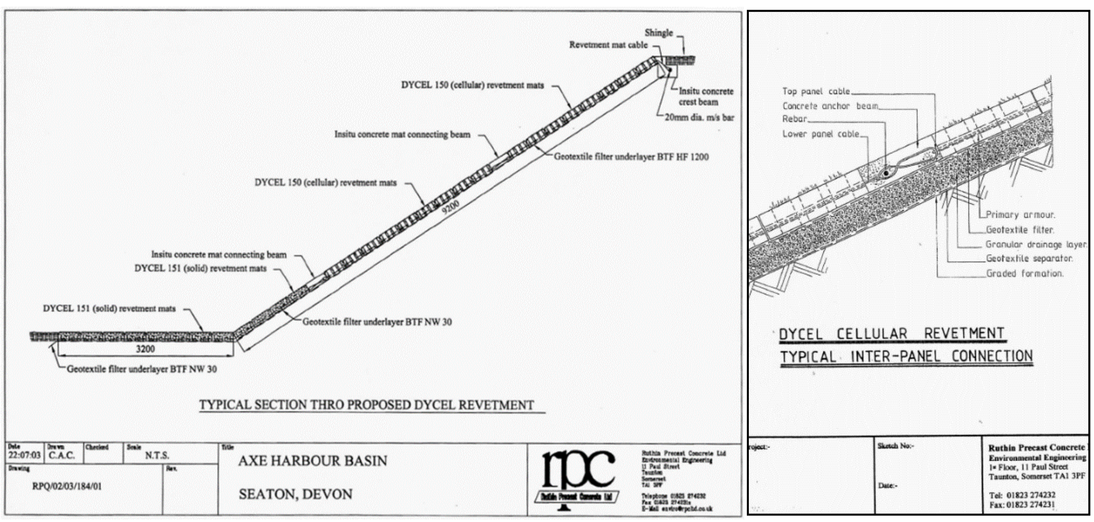


Figure 2-12 Typical Section through Proposed Dycel Revetment and Typical Inter-Panel Connection
Source: Axmouth Harbour Management (2003)

Figure 2-13 2009 The basin at L.W with A & B Pontoons
Source: AYC (no date)

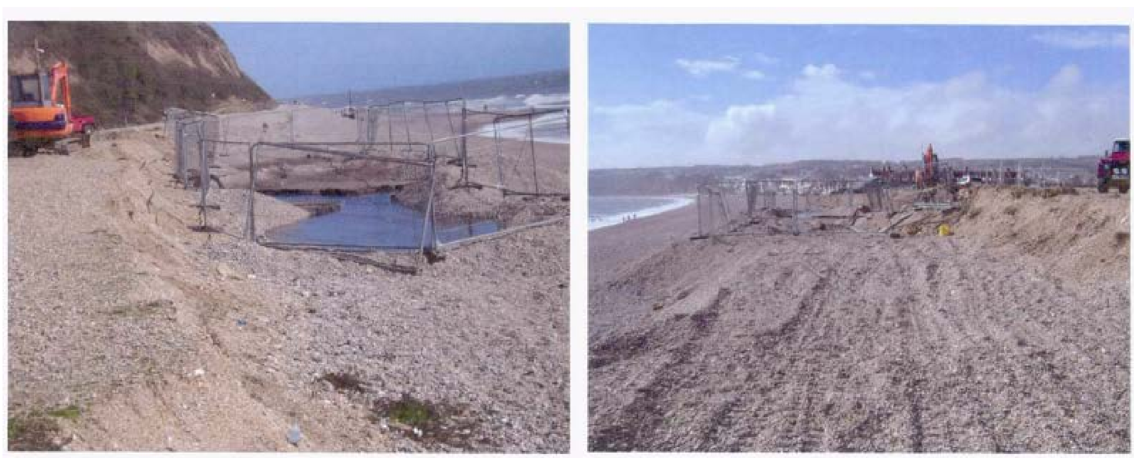


Figure 2-14 Trench dug in 2009 for dredging discharge material
Source: Author unknown 1 (no date)

2.6 River Axe East Bank

Along the east bank of the River Axe forms part is a quay wall used by local registered fishermen. Details of the known changes to the structure are described below:

- In 1978 the original harbour arm was constructed as a mass concrete gravity structure surrounded by sheet piling.
- In 1985 the harbour wall was refurbished (Royal Haskoning, 2015).
- Between 1998 and 2001, refurbishment of the Axmouth Harbour walls was undertaken and a new 20m Harbour Arm extension was constructed. The extension at the river mouth consisted of a lower level cellular structure, backfilled with concrete and supported by timber fendering and breastwork. The design was undertaken by Posford Duvivier and the general arrangement is shown in Figure 2-15 and Figure 2-16.

- In 2003 repairs to the steel sheet piling that formed the harbour arm were undertaken, along with treatment to small areas of Accelerated Low Water Corrosion (ALWC) (Royal Haskoning 2015).
- In 2005 there was an inspection of the harbour arm which identified ALWC across a broader area than previously recorded. No work was identified for scour repair (Royal Haskoning, 2015).
- In 2006 an ALWC inspection was commissioned by EDDC, and undertaken by Royal Haskoning. The investigation concluded there was little risk to the harbour walls stability, and recommended a programme of repeat surveys to monitor the condition (Royal Haskoning, 2015).

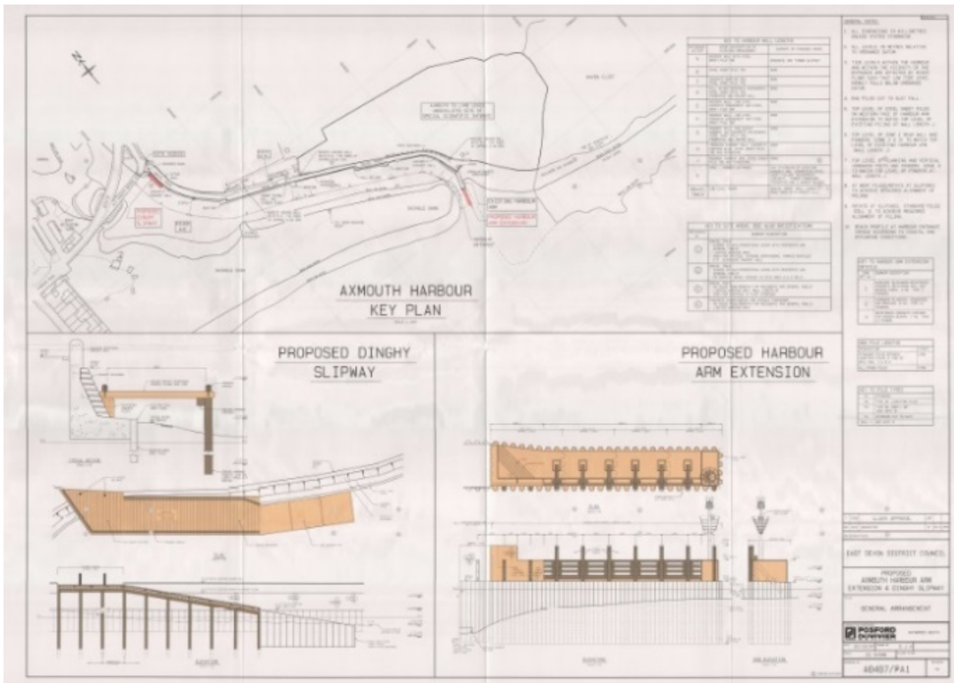


Figure 2-15 Axmouth Harbour Arm extension plans
Source: Posford Duvivier (1997)

Figure 2-16 Axmouth Harbour Arm extension Source: Axe Harbour Management Company (1997)

Condition Assessment

A visual inspection and condition assessment of the defences along the BMP frontage, between Seaton Hole to the east bank of the River Axe at the river mouth was undertaken by CH2M's coastal engineers on the 14th June 2017 to determine their condition and residual life. For the purpose of this assessment, the BMP frontage was divided into thirteen sections defined by the flood defence or coastal protection measure present, as shown in Figure 3-1.

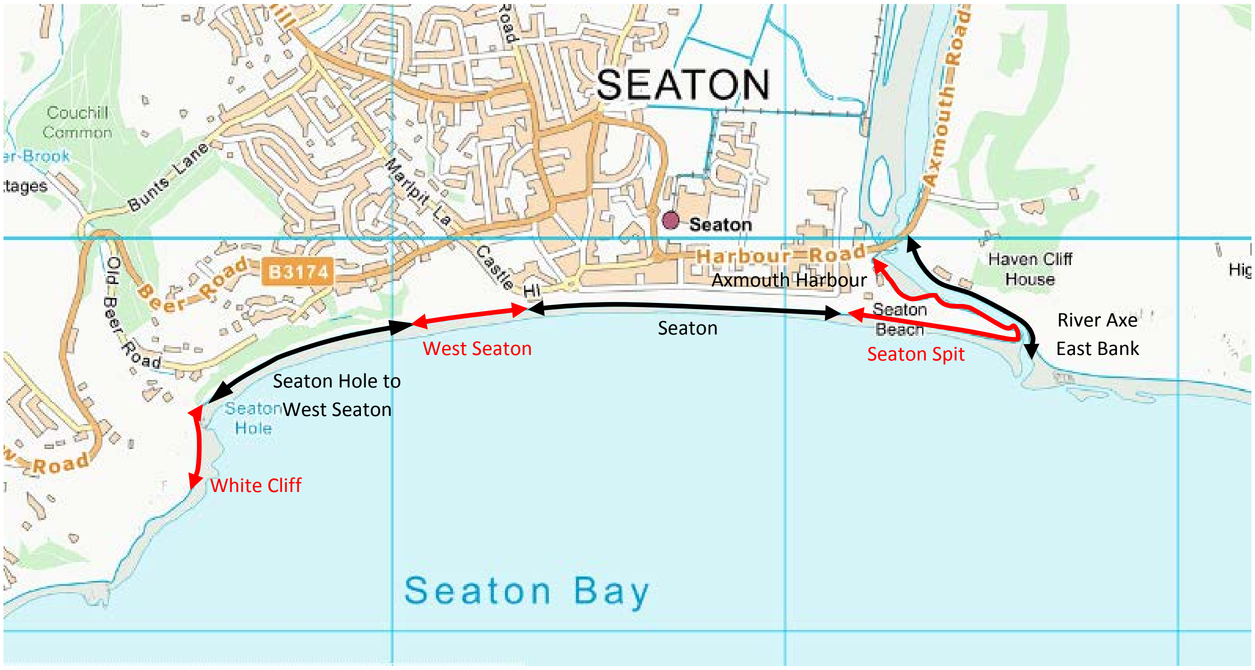


Figure 3-1 Condition assessment frontages within the study area

Basemap Copyright: Ordnance Survey (2017)

Table 1 Description of condition assessment frontages

Frontage	Frontage Name	Approx. Length (m)	Defence Description
1	White Cliff	n/a	High chalk cliffs
2	Seaton Hole to West Seaton	25	Mass concrete protecting a rock armour core
3		470	Intermittently placed rock armour protecting cliffs
4		5	Composite defence comprising rock and concrete gabions
5		65	Sprayed concrete seawall
6	Seaton Flood Defences	375	Stone block seawall
7		785	Concrete seawall
8	Seaton Spit	440	Axe Yacht Club beach area
9		250	Inner spit on the River Axe west bank
10		290	Axmouth Harbour
11	River Axe East Bank	65	Stone block wall 1
12		130	Steel sheet pile wall
13		225	Stone block wall 2

It is noted that there was a significant amount of debris on the beach areas as a result of cliff erosion to the west of Seaton. This is a health and safety hazard for members of the public accessing the beach and cliff areas, and the risk of further incidents should be considered alongside the specific findings in the following sections.

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, as presented below. The Environment Agency's Asset Deterioration Guidance was then used to assess the residual life of each structure, as described below.

Condition Assessment Key Features

Table 2 describes the five general condition grades that range from 'Very Good' to 'Very Poor'.

Table 2 General condition grades for structures in accordance with the Environment Agency's CAM
Source: Environment Agency (2012)

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

Condition Assessment Specific Features

The following tables show the condition grades (including rating and key features) for the structures that exists along the BMP frontage, sourced from the CAM (Environment Agency, 2012) and used for the visual inspection and condition assessment. These were used in addition to the general descriptions shown in Table 2.

Table 3 Concrete seawall grading key features

Source: Environment Agency (2012)

Grade	Rating	Description
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 4 Rock revetment key features

Source: Environment Agency (2012)

Grade	Rating	Description
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 5 Concrete revetment key features

Grade	Rating	Description
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 6 Beach key features

Grade	Rating	Description
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.

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.

The models incorporate three different maintenance regimes and three deterioration categories which are defined in Table 7 and Table 8.

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

Table 7 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 8 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)

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).

3.1 Frontage 1: White Cliff

Asset Description: Very high cliffs with spalled rock at the cliff base causing a high and steeply sloped toe. There were few signs of recent erosion of the cliff or slip falls, with significant vegetation growth at the cliff toe. Some large rocks at the base of the cliff provide a small degree of protection, although there are too few rocks to create a formal defence profile.

Well-vegetated area of spalled rock at cliff base

Signs of recent slips with large and small rock

Condition Description: Rocks at base provide some protection to the cliffs, although the rocks are irregularly placed. Cliffs included several cracks, which have previously led to rock of varying sizes breaking away from the cliff, providing feed to the beach.

Condition Grade: n/a

Residual Life: n/a

3.2 Frontage 2: Seaton Hole to West Seaton – Concrete Revetment

Asset Description: Very narrow section of gabion baskets filled with concrete-filled sand bags, intended to provide protection against erosion for the outfall discharge. Adjacent to flood defence of rock armour core and thin unreinforced concrete cover (approximately 100-300mm thick). Approximate defence slope of the concrete and rock structure was 1 in 2.

Gabion baskets protecting an outfall

Undermining and outflanking have caused slumping

Steep, concrete revetment surface was cracked

Failure at slope toe leading to loss of fill

Condition Description: Gabion baskets were in good condition, with no apparent damage or gaps, although outflanking and undermining at the toe of the structure has allowed loss of underlying material and settlement of the structure.

Some large cracks within the adjacent concrete structure indicated potential settlement. One very large failure (approximately 10m width) of the structure included loss of concrete cover, which allowed the rock core to be washed out. Relatively new handrails at the crest were in good condition, although the footpath passed directly behind the main structure, and may be unstable due to the failing defence.

Condition Grade: Grade 4 – POOR

Residual Life: As the structure has deteriorated to Grade 4, it is already experiencing significantly reduced performance. Assuming the guidance values for a narrow permeable revetment in the coastal zone for 'Sloping walls with slope protection or revetment', the best estimate for complete performance failure is 10 years. Table 9 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 9 Frontage 2 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	0 years	9 years
Medium	**0 years**	**10 years**
High	0 years	20 years

3.3 Frontage 3: Seaton Hole to West Seaton – Rock Armour

Asset Description: A relatively long section of rock defences at the toe of the eroding cliff. Various sections of rock defences were distributed along the frontage comprising:

- 3-6t rock of various sources, including some broken concrete, which appeared to be placed in a single layer;
- Approximately 5-8t rock on single source, which appeared to be placed in multiple layers; and
- Narrow rock armour buried within a narrow and steeply-sloped beach.

Recent erosion of the cliffs was observed in several locations, although well-established vegetation behind the rock armour was also noted in some places. The cliffs behind the rock structures showed signs of groundwater percolation out of the cliff. Multiple outfalls were present in the upper cliff area, although it was not clear whether there was active discharge from these structures. At the eastern end of the frontage, the beach was narrower, with a steeply-sloping gravel beach overlying the rock structure.

Generally consistent profile

Large voids present within the structure

Rock revetment buried in some areas as beach levels increase in elevation

Signs of recent cliff failure behind buried rock revetment

Condition Description: Characterised by an uneven profile with fluctuating crest level and crest width between each section of rock defence. No signs of undermining of the individual flood defence lengths ensuring no slumping or defence failure. The rock armour was sparsely placed in some areas, allowing more direct wave action on the cliff base. No evidence of geotextile material below the rock armour where inspection was possible (as-built drawings suggest single sided revetment was placed directly on to bedrock). A limited number of large voids, or displacement of rock within the rock armour structures were observed. Several areas indicated recent cliff collapses, providing feed material to the beach. The western half of the defence was in better condition, with a more uniform profile, and fewer missing armour rocks.

Condition Grade: Grade 4 – POOR (although local areas with much higher condition)

Residual Life: The structure has deteriorated to Grade 4, therefore it is already experiencing significantly reduced performance. Assuming the guidance values for a narrow, permeable, coastal revetment, the best estimate for complete performance failure is 10 years. Table 10 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 10 Frontage 3 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	0 years	9 years
Medium	**0 years**	**10 years**
High	0 years	20 years

3.4 Frontage 4: Seaton Hole to West Seaton – Composite Gabion Defence

Asset Description: A complicated structure with multiple material types, creating a composite coastal erosion protection structure. The cliff toe was exposed behind the main beach area. This was overlain by multiple layers of concrete which have been eroded and undermined. Rock gabions and concrete-filled sand bag gabions provided protection to the upper section of cliff, although the access steps and former outfall behind the defence were out of use.

Rock at toe protection concrete slab and gabions

Deformation of gabion baskets and loss of fill material

Condition Description: Undermining and outflanking of the defence was noted. The surface of the concrete elements appeared to have several locations of more intense surface damage, although these did not appear to coincide with exposed reinforcement. The rock-filled gabions were deformed, with corrosion of gabion baskets in several locations, particularly at the crest. Some gaps within the gabion baskets were also observed, allowing the loss of rock fill. Newer concrete-filled sand bag gabions were in very good condition, with no deformation or signs of basket corrosion.

Condition Grade: Grade 4 – POOR

Residual Life: As the structure has deteriorated to Grade 4, it is already experiencing significantly reduced performance. Assuming the guidance values for permeable coastal 'Sloping walls with slope protection', the best estimate for complete performance failure is 10 years. Table 11 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 11 Frontage 4 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	0 years	9 years
Medium	**0 years**	**10 years**
High	0 years	20 years

3.5 Frontage 5: Seaton Hole to West Seaton – Sprayed Concrete Wall

Asset Description: Concrete toe protecting a newer section of sprayed concrete seawall. The structure included a significant number of drainage channels to reduce saturation of the soil and cliff behind the wall. The overall structure was relatively narrow, with some gravel placed behind the wall and the cliff.

Sprayed concrete upper wall and reinforced concrete toe

Spalling of surface concrete resulted in exposure of underlying steel

Vertical cracking aligned to drainage channels

Patch repairs to concrete toe

Condition Description: Very shallow cover to the steel reinforcement had caused rust staining in several areas, and has also lead to exposure of the corroded reinforcement as a consequence of saline penetration and surface spalling. Some vertical surface cracking was observed, and generally coincided with the line of drainage holes. The concrete toe was in good condition, although some minor cracking was also recorded, along with areas of recent patch repair to address previous damage to the structure.

Condition Grade: Grade 3 – FAIR

Residual Life: Assuming the guidance values for a concrete vertical wall in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 15 years. The duration for the defence to deteriorate to a condition of complete performance failure is 30 years. Table 12 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 12 Frontage 5 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 Frontage 6: Seaton Town – Stone Block Seawall

Asset Description: The king cliff areas were protected by the main seawall structure and an approximately 10m wide promenade. The main seawall at the western part of the Seaton sea defences was composed of a near-vertical stone block seawall, with a concrete wave return at the crest of the wall. The block wall was made of relatively small stone of consistent size (approximately 0.3m square and rectangular stone). The narrow fronting gravel beach formed a steep upper slope.

The frontage also included several access points, comprising ramps and stepped section. There were formed of concrete and stone block walls, with stainless steel handrails offering support along the structure.

Stone block seawall topped with concrete recurve

Patch repairs to grouting between stone blocks

Localised loss of grouting between stone blocks

Typical stepped access in this frontage

Condition Description: The cliffs were well-protected and show no recent signs of erosion. The stone block seawall was in very good condition, with no indication of surface erosion or cracking. Mortar was in good condition, although multiple locations showed recent patch repairs where damage was previously recorded. There was no observed loss of stone blocks or fill material from within the wall, although some isolated locations displayed missing mortar which might lead to more significant structural damage if not addressed. Stainless steel handrails were in good condition and showed few signs of deformation or corrosion.

The reinforced concrete in the public access infrastructure was in good condition, with no significant cracking. Some minor rust staining was observed, although there were no signs of exposed reinforcement or spalling of concrete. The stainless steel handrails were in good condition with no signs of deformation or corrosion. The grouted stone protection at the access infrastructure was in good condition, with no damage or loss of grouting.

Condition Grade: Grade 2 – GOOD

Residual Life: Assuming the guidance values for a vertical brick/masonry wall in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 40 years. The duration for the defence to deteriorate to a condition of complete performance failure is 55 years. Table 13 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 13 Frontage 6 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

3.7 Frontage 7: Seaton Town – Concrete Seawall

Asset Description: A reinforced concrete recurved seawall formed the upper section of the flood defences, fronted by an approximately 5m wide asphalt promenade, providing amenity access along the flood defences. A reinforced concrete wave-return wall protects the promenade as the lower section of the defence, and acts as a landward barrier for the gravel beach.

The frontage also included several access points, comprising concrete steps. Each access point generally coincided with a stainless steel flood gate that can be closed during extreme events to provide a consistent defence level in the upper flood wall. Each gate structure included stainless steel seatings and rubber seals. This section also included double gates at the main slipway.

Vertical cracking of concrete seawall with wave return

No signs of wall undermining, or slumping

Beach levels increase further east

Multiple flood gates present, with damage to rubber seals

Condition Description: The upper seawall was in good condition with no apparent defects. Expansion joints were well maintained, with replacement of material at several locations. The reinforced concrete seawalls showed some signs of surface abrasion and rust staining, although further damage was not noted.

Each flood gate included large patch repairs (additional material riveted to the surface) to previous damage, and some minor surface corrosion, particularly at the hinges. The rubber seal on the outer section of the gates was either cracked or entirely detached from the gate, affecting the ability to achieve a complete seal.

Condition Grade: Grade 2 – GOOD

Residual Life: Assuming the guidance values for a vertical concrete wall in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 50 years. The duration for the defence to deteriorate to a condition of complete performance failure is 70 years. Table 14 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 14 Frontage 7 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	35 years	45 years
Medium	**50 years**	**70 years**
High	65 years	95 years

3.8 Frontage 8: Seaton Spit – Axe Yacht Club Beach Area

Asset Description: A very wide gravel beach area provides the flood and erosion protection to the Axe Yacht Club and the Axmouth Harbour area. The beach was composed of natural gravel, overlying dredged material from the river and harbour area. The end of the spit appeared to be a more natural gravel structure, with well-graded beach material and a number of storm ridges.

Wide, shallow-sloped beach fronting Axe Yacht Club

Beach steepens further east, exposing consolidated silt within the beach core (previous dredged material)

Steep upper beach in previous disposal area for dredging

Cliffing was noted at the toe of the upper beach, exposing consolidated silt

Condition Description: The wide shallow gravel beach in the western portion of this section provides a high crested natural flood defence. The beach was steeper in the eastern portion of this frontage, although this has led to exposure of more highly-compacted, fine-grained, dredged sediment. Exposure of the underlying dredged material has permitted cliffing of the beach to occur, and has made accessing the lower beach area more difficult.

Condition Grade: Grade 3 – FAIR

Residual Life: Assuming the guidance values for a shingle beach without beach control structure in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 20 years. The duration for the defence to deteriorate to a condition of complete performance failure is 45 years. Table 15 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 15 Frontage 8 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

3.9 Frontage 9: Seaton Spit – Spit River Bank

Asset Description: Dredged material has been deposited on the spit to create a high bank at the back of the beach. The lower bank was composed of more natural gravel material, and formed a more shallowly-sloping foreshore than the upper bank. Further upstream the foreshore transitions to steeper slopes with cobbles and gabion rock mattresses.

Natural spit formation at the mouth of the River Axe

The back of the spit formed of a steeply-sloped gravel beach

Larger cobbles protecting the back of the spit where cliffing and erosion has occurred

Damage to grass cover and underlying gabion mattresses

Condition Description: The fill material of the upper bank has been eroded over time, leading to significant cliffing along this entire frontage, with larger pieces of rock, brick and concrete waste being exposed within the bank. Some vegetation has colonised the upper bank, although this does not appear to provide significant stability to the bank. The lower beach area is in generally good condition, with the gravel forming a natural structure. The gabion rock mattress at the junction between this frontage and the adjacent Axmouth Harbour has become exposed with signs of bank erosion and steepening of the bank at the toe.

Condition Grade: Grade 3 – FAIR

Residual Life: Assuming the guidance values for a shingle beach without beach control structure in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 20 years. The duration for the defence to deteriorate to a condition of complete performance failure is 45 years. Table 16 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 16 Frontage 9 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

3.10 Frontage 10: Seaton Spit – Axmouth Harbour

Asset Description: Stabilisation of the banks in the main harbour area was provided by moderately-sloped rock gabion mattresses along the lower bank. The upper bank appeared to be formed of gravel and dredged material, forming a steeper bank, with some superficial vegetation along the crest and the mid-slope.

Harbour area includes gabion mattresses in the intertidal area

Some damage to steel has allowed loss of fill material

The benign wave conditions have allowed vegetation growth

Gabion structures forming steeper defence profile

Condition Description: Some damage was noted to the gabion mattresses along the lower slope, leading to the potential loss of rock fill. Despite this, there was no significant loss of material, and no slumping of the gabion mattresses was observed. The intertidal area of the lower slope was partly covered by fine gravel and other sediment, resulting in burial of much of the gabion structures. The upper gravel slope had a generally consistent profile, with fewer signs of cliffing than was present in the adjacent Frontage 9. The vegetation established at the crest and the mid-slope may provide some stability to these areas.

Condition Grade: Grade 2 – GOOD

Residual Life: Assuming the guidance values for a wide permeable revetment in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 35 years. The duration for the defence to deteriorate to a condition of complete performance failure is 45 years. Table 17 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 17 Frontage 10 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	22 years	30 years
Medium	**35 years**	**45 years**
High	40 years	60 years

3.11 Frontage 11: River Axe Eastern Bank – Stone Wall 1

Asset Description: A near-vertical composite structure comprised eastern bank of the River Axe, downstream of the B3172 bridge. The lower wall was formed of steel sheet piles, which were mostly submerged during the site visit. The middle section of the flood defence was a very steeply-sloped, grouted rock wall, while the upper section of the defence was a vertical reinforced concrete wall.

Composite flood defence with timber slipway

Composite structure with stone block lower wall and concrete upper wall

Surface corrosion of steel sheet pile toe

Some vertical cracking of the upper concrete wall

Condition Description: Recent surveys indicated that the steel sheet piles may have suffered from accelerated low water corrosion (ALWC), although this could not be observed during the site visit. The condition of the grouted stone wall was variable along its length, with deteriorating condition to the south. Some signs of damage to grouting and occasional gaps were observed, suggesting loss of rock units had occurred. The condition of the vertical concrete wall was variable along its length, with some vertical cracking affecting the overall integrity of the upper wall. No overall slumping or heave was noted, and no lateral bulging of the structure was observed, suggested that there were no issues with undermining or voids behind the structure.

Condition Grade: Grade 2 – GOOD

Residual Life: Assuming the guidance values for a vertical brick/masonry wall in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 50 years. The duration for the defence to deteriorate to a condition of complete performance failure is 70 years. Table 18 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 18 Frontage 11 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	35 years	45 years
Medium	**50 years**	**70 years**
High	65 years	95 years

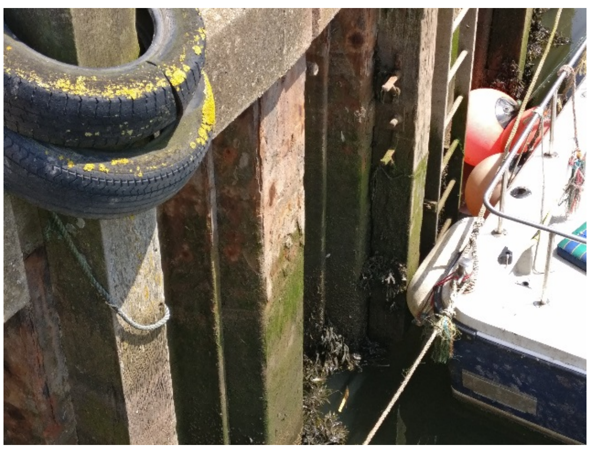
If the steel sheet pile is assumed to be in worse condition due to ALWC and assigned condition Grade 3 – Fair, the structure would have a lower overall residual life. Assuming the guidance values for a cantilevered sheet pile in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 25 years. The duration for the defence to deteriorate to a condition of complete performance failure is 35 years.

3.12 Frontage 12: River Axe Eastern Bank – Steel Sheet Pile Wall

Asset Description: The flood defences in this frontage were composed of a high steel sheet piled wall, capped with a narrow concrete capping beam and a short concrete flood wall. Several timber posts distributed along the wall provided fendering for vessels accessing the structure, but provide little in the way of structural stability.

Steel sheet pile wall creates vertical defence

Concrete slab and short flood wall



Surface corrosion of the main sheet pile wall

Surface corrosion in the upper section of the sheet pile

Condition Description: The steel sheet pile wall showed signs of substantial corrosion, with some areas of surface damage suggesting significant penetration through the structure. Recent survey work also indicated that the steel sheet piles have suffered from ALWC, although some of these areas could not be observed during the site visit due to tide level. The concrete capping and short flood wall were in generally good condition, although in some isolated areas the wall had been repaired after previous damage.

Condition Grade: Grade 3 – FAIR

Residual Life: Assuming the guidance values for cantilevered sheet pile in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 25 years. The duration for the defence to deteriorate to a condition of complete performance failure is 35 years. Table 19 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 19 Frontage 12 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	15 years	25 years
Medium	**25 years**	**35 years**
High	25 years	35 years

3.13 Frontage 13: River Axe Eastern Bank – Stone Wall 2

Asset Description: A near-vertical composite structure provides the flood defence for the eastern bank towards the river mouth, with a structure similar to the defence described in the 'River Axe Eastern Bank – Stone Wall 1' frontage. The lower wall was formed of steel sheet piles, which were mostly submerged during the site visit. The middle wall section was grouted rock, forming a steeply sloping seawall, while the upper wall is a short vertical concrete wall.

Stone block wall along the River Exe east bank

Patch repairs to the upper section of stone block wall

Patch repairs to the concrete flood wall

Concrete platform connected to the seawall close to the river mouth

Condition Description: Recent surveys indicated that the steel sheet piles have suffered from ALWC, although this could not be observed during the site visit. The grouted stone wall was broadly in good condition, with no area of obvious damage or cracking in the surface of the structure. No overall slumping or heave, and no lateral bulging of the structure suggested that there were no undermining and issues with voids or drainage behind the structure. Several sections of the upper concrete flood wall have been repaired to ensure that the wall provides a consistent standard of flood protection, although there were signs of damage that had not been repaired.

Condition Grade: Grade 2 – GOOD

Residual Life: Assuming the guidance values for a vertical brick/masonry wall in the coastal environment, the best estimate for the defence to deteriorate to a condition of significantly reduced performance is 50 years. The duration for the defence to deteriorate to a condition of complete performance failure is 70 years. Table 20 shows the best estimates within the medium maintenance regime alongside estimates for low and high maintenance regimes.

Table 20 Frontage 13 estimated performance values

Maintenance Regime	Significantly Reduced Performance	Complete Performance Failure
Low	35 years	45 years
Medium	**50 years**	**70 years**
High	65 years	95 years

3.14 Summary of Defence Condition

Table 21 summarises the assessed defence condition and residual life of each coastal defence element along the 13 sections of frontage described in Sections 3.1 to 3.13.

Table 21 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	High chalk cliffs	n/a	n/a	n/a
2	Mass concrete protecting a rock armour core	Grade 4 (Poor)	0 years	10 years
3	Intermittently placed rock armour protecting cliff	Grade 4 (Poor)	0 years	10 years
4	Composite defence comprising rock and concrete gabions	Grade 4 (Poor)	0 years	10 years
5	Sprayed concrete seawall	Grade 3 (Fair)	15 years	30 years
6	Stone block seawall	Grade 2 (Good)	40 years	55 years
7	Concrete seawall	Grade 2 (Good)	50 years	70 years
8	Axe yacht Club beach area	Grade 3 (Fair)	20 years	45 years
9	Inner spit on the River Axe west bank	Grade 3 (Fair)	20 years	45 years
10	Axmouth Harbour	Grade 2 (Good)	35 years	45 years
11	Stone block wall 1	Grade 2 (Good)	50 years	70 years
12	Steel sheet pile wall	Grade 3 (Fair)	25 years	35 years
13	Stone block wall 2	Grade 2 (Good)	50 years	70 years

SECTION 4

Overtopping Analysis

To better understand the flood/breach risk within the study area, it is necessary to know the levels of wave overtopping discharge that can occur along the BMP frontage. Wave overtopping analysis was completed as part of the Lyme Bay Tidal Procedures Project in 2015 (CH2M, 2015b), and has been adopted within this project to establish the standard of protection that the current defences afford in the present day, and how it may change in the future.

4.1 Methodology

The overtopping analysis was undertaken in accordance with best-practice guidance of the time, contained within the Wave Overtopping of Sea Defences and Related Structures: Assessment Manual (Environment Agency, 2007). This has subsequently been superseded by a newer version of guidance (Environment Agency, 2017), however the results of these analytical approaches are very similar, and may be expected to produce similar results.

The wave overtopping analysis work was completed using the Neural Network tool, allowing the examination of a larger number of design conditions. While this tool was developed to allow the appraisal of structures rather than shingle beaches, it has been adopted as an important tool for examining large numbers of wave conditions for multiple defence types. Sensitivity testing was conducted to address any potential issues with the evolution of the beach profile throughout a design storm event. This testing indicated that overtopping discharge rates during a 0.5% AEP (1 in 200-year) event may double as a consequence of adopting an emergency profile in place of the baseline profile, however the emergency profile used does not include the gravel berm that formed under all previous storm conditions and which plays a significant role in dissipating wave energy. As such the baseline profile was considered a reasonable representation of the beach.

4.1.1 Overtopping Tolerances

The overtopping guidance includes the overtopping limits for the safety of people and vehicles, and overtopping limits for the structural design of flood defences. These limits are dependent on the incident wave height as described in Table 22 and Table 23.

Table 22 Limits for overtopping for people and vehicles
Source: Environment Agency (2017)

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	0.3	600
H_{m0} = 2 m	1	600
H_{m0} = 1 m	10-20	600
H_{m0} < 0.5 m	No limit	No limit
Cars on seawall / dike crest, or railway close behind crest		
H_{m0} = 3 m	<5	2000
H_{m0} = 2 m	10-20	2000
H_{m0} = 1 m	<75	2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Table 23 Limits for overtopping for structural design of breakwaters, seawalls, dikes and dams
Source: Environment Agency (2017)

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

The latest guidance (Environment Agency, 2017) also states that buildings or apartments are frequently constructed immediately shoreward of the flood defences or behind a coastal boulevard or promenade. In such cases, the mean wave overtopping discharges should be limited to 1 l/s/m. This arrangement is broadly the case at Seaton, where several properties are located landward of the Promenade, and will be susceptible to inundation during overtopping events.

4.1.2 Cross-Shore Profiles

Previous overtopping work for Seaton examined wave overtopping at four representative locations, as shown in Figure 4-1. These locations have characteristic cross-shore profiles of the frontages which would contribute to the flooding of Seaton during extreme events.

- Slipway – The slipway profile provides data for the main slipway in Seaton, which is a key weak point if the flood gates are not closed. The slipway provides a smooth, impermeable surface for wave run-up of incident waves and potentially large overtopping.

- 6a01177 – The profile provides a representation of flood defences with a narrow fronting beach.
- 6a01169 – The profile provides a representation of flood defences with a wider fronting beach.
- 6a01163 – The profile provided data for the original project to asses flooding in the River Axe and the main Axmouth Harbour area, where the separate and combined tidal and fluvial flood risks were examined.

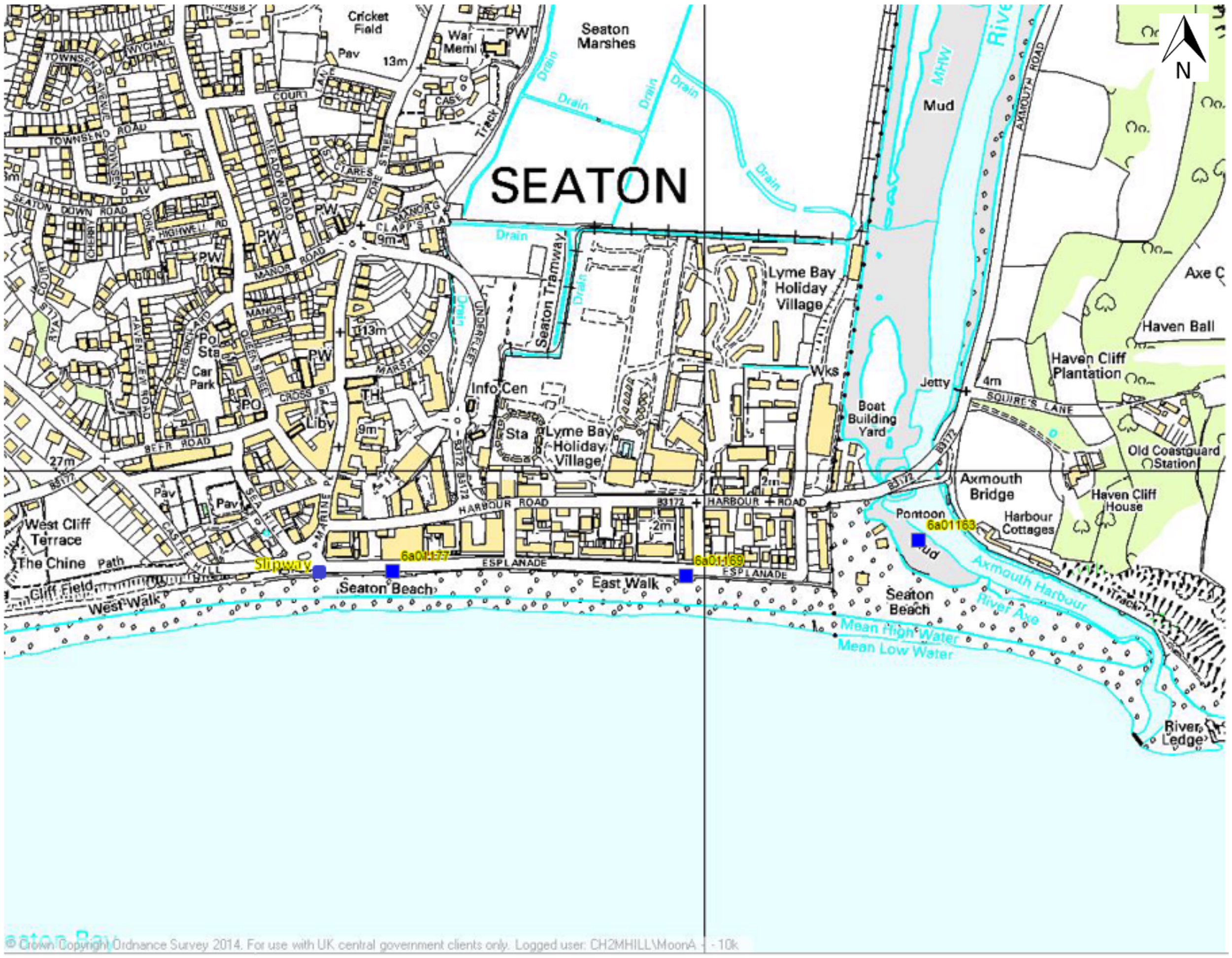


Figure 4-1 Map showing the four profile locations assessed at Seaton
Crown Copyright Ordnance Survey (2014)

Overtopping of the Stone Block Seawall defence between The Hideaway Café and the concrete seawall at the base of Castle Hill would not pose a direct flood risk to the town. Overtopping discharge would be prevented from propagating inland and affecting properties by elevated road infrastructure between existing flood walls. Access along the backing promenade to the West of the main Seaton flood defences may be affected by overtopping, and there is potential for wave action to affect cliff stability to the west of Seaton, although no property damage is to be expected from wave overtopping.

Generalisation of previous survey data was completed to produce a representative cross-shore profile and allow overtopping discharge to be calculated in accordance with Environment Agency guidance (2007). The generalised profiles used in the analysis include a lower slope, berm, upper slope, and seawall. Figure 4-2 indicates the outcome of this process for profile 6a01177.

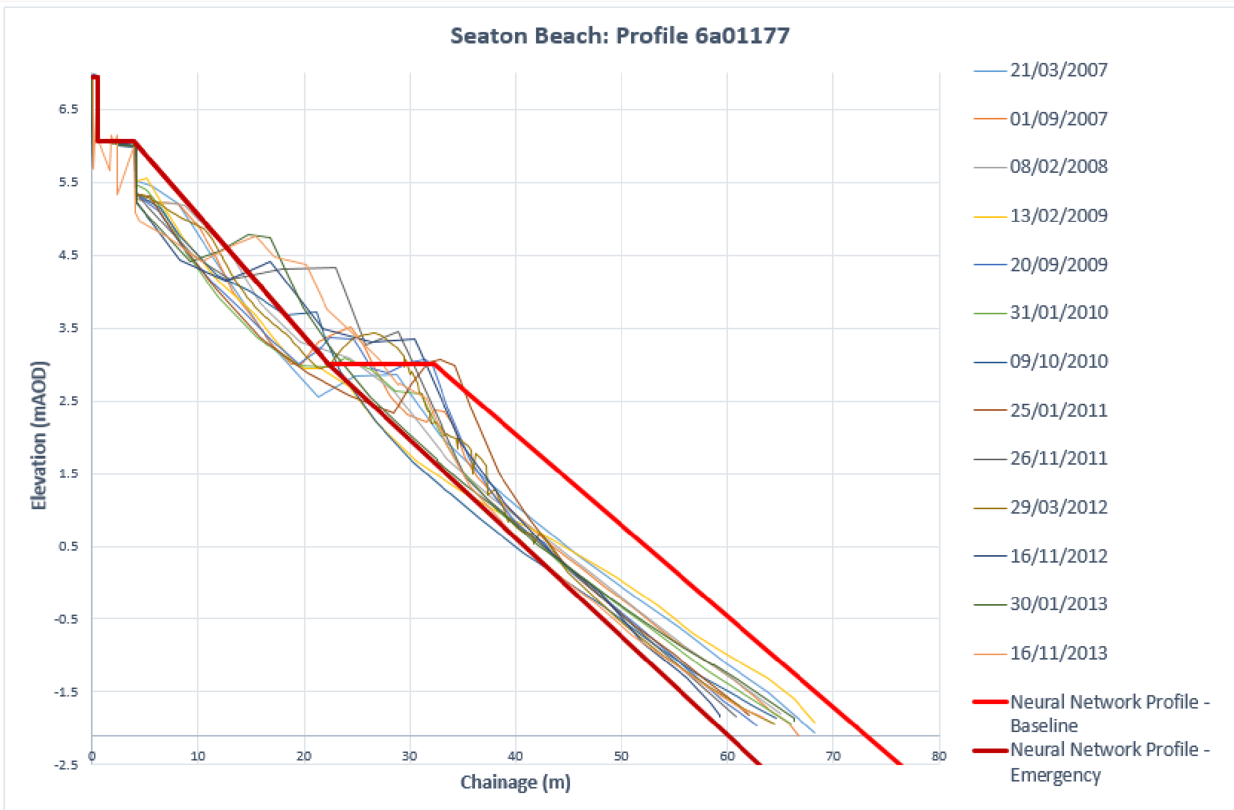


Figure 4-2 Example generalised profile, derived for neural network analysis at profile 6a001177

Simplification of the flood defence at Seaton to was necessary due to the presence of the upper and lower walls. This was included within the EurOtop beach profile by increasing the crest level of the upper slope to the crest of the lower seawall. The steepness of the upper and lower slopes of the generalised Baseline profile provided reasonable representations of the upper and lower slopes of the actual beach, while the berm in the centre of the beach is typical of many profiles. The promenade and upper seawall are accurately represented in the generalised profile.

4.1.3 Water Levels and Wave Conditions

The Coastal Flood Boundary data set was used to derive the extreme water levels in accordance with Environment Agency guidance (Environment Agency, 2011), adopting Point 20 as the most appropriate wave data point for Seaton. Extreme wave heights were derived from Environment Agency data (Environment Agency, 2012), with Point 31 (gl2632) identified as most appropriate due to location south of Seaton. JPA analysis was completed on the wave height and water level data to identify conditions with a range of marginal joint probabilities. Wave periods for each extreme significant wave height were derived from the wave period probability table associated with the data source (Environment Agency, 2012).

SWAN modelling was previously completed to transform several combinations of a discrete number of significant wave height, peak wave periods, still water levels and wind forces from offshore to the nearshore. The joint probability extreme offshore wave data to be used in design was transformed inshore by interpolating between the most closely matched SWAN conditions (Table 24).

Table 24 SWAN model offshore conditions

Significant Wave Height, H_s (m)	Peak Wave Period, T_p (s)	Still Water Level (mOD)	Wind Force
2	4	1.85	0
4	8	2.15	2
6	12	2.50	4
8	16	2.80	6
10	20	3.10	8
12		3.40	10
			12

Preliminary analysis by CH2M (2015b) identified that the waves originating from the south and southwest direction sectors produced the worst-case overtopping, whilst wave conditions originating from the southwest sector resulted in lower overtopping rates and would therefore not be required carried forward for the full overtopping analysis.

4.1.4 Climate Change Allowance

The previous analytical work accounted for the effects of climate change through the consideration of UKCP09 data. Local sea level rise allowances were established for the time duration between the baseline data and 2070, and the baseline data and 2115. These elevated water levels allowed modification of the offshore wave data, which was transformed inshore through the same process of interpolation of the SWAN data. This data was then used to recalculate wave overtopping discharge rates at the defences.

4.2 Results

The outputs of the overtopping analysis are presented in Table 25, summarising the worst-case flooding for each return period and each profile. As the Seaton flood defences are exposed to wave heights between 2m and 3m, the overtopping threshold for people and vehicle safety is defined as 0.3 l/s/m (Table 22) and the threshold for flooding of properties behind the seawall of 1 l/s/m. Finally, an approximation can be made for the structural design overtopping threshold by assuming the structure is equivalent to a breakwater with rear side designed for wave overtopping, with a threshold of 5 l/s/m.

Table 25 Overtopping analysis results for Seaton – present day

Source: CH2M (2015b). Values in l/s/m. Green – discharge less than 0.3 l/s/m; Yellow – discharge 0.3-1.0 l/s/m; Orange – discharge 1.0-5.0 l/s/m; and Red – discharge greater than 5.0 l/s/m.

Profile	Event Return Period (1 in X)								
	2	10	20	30	50	75	100	200	1000
Slipway No Gate	1.02	1.25	1.86	1.99	2.21	2.37	2.48	2.87	4.64
Slipway With Gate	0.41	0.48	0.72	0.78	0.85	0.91	0.96	1.08	1.79
6a01177	0.61	0.73	1.11	1.2	1.32	1.43	1.5	1.72	2.90
6a01169	0.35	0.56	0.73	0.76	0.81	0.85	0.87	0.96	1.33

The present day scenario indicates that the flood defences in narrow beach areas (represented by Profile 6a01177) will exceed the overtopping threshold during 5% AEP (1 in 20-year) events. The equivalent threshold is not exceeded by the wider beach areas (represented by Profile 6a01169) until a 0.1% AEP (1 in 1,000-year) event. With the gate closed and effectively sealed, the overtopping rate is not exceeded until the 0.5% AEP for damage to properties behind the coastal promenade. If the gate is not closed, this threshold will be exceeded in events more frequent than a 50% AEP event. There were no tested conditions in the present day when the defences appear to be at risk of structural failure during a storm event.

Some work was also completed to examine the change in wave overtopping discharge as a consequence of anticipated climate change, however the full range of return periods was not examined. The climate change analysis instead focussed on the changes during the most extreme events, with 0.5% AEP (1 in 200-year) and 0.1% AEP (1 in 1,000-year) examined for 2070, and 0.1% AEP examined in 2115 (Table26 and Table 27).

Table26 Climate change analysis outputs for 0.5% AEP (1 in 200-year) tests

Source: CH2M (2015b). Values in l/s/m. Green – discharge less than 0.3 l/s/m; Yellow – discharge 0.3-1.0 l/s/m; Orange – discharge 1.0-5.0 l/s/m; and Red – discharge greater than 5.0 l/s/m.

Profile	RP (1 in 200)	
	Present day	2070
Slipway No Gate	2.87	6.24
Slipway with Gate	1.09	1.46
6a01177	1.72	3.76
6a01169	0.96	1.46

Table 27 Climate change analysis outputs for 0.1% AEP (1 in 1,000-year) tests

Source: CH2M (2015b). Values in l/s/m. Green – discharge less than 0.3 l/s/m; Yellow – discharge 0.3-1.0 l/s/m; Orange – discharge 1.0-5.0 l/s/m; and Red – discharge greater than 5.0 l/s/m.

Profile	RP (1 in 1,000)		
	Present day	2070	2115
Slipway No Gate	4.64	8.49	14.76
Slipway with Gate	1.79	3.18	5.87
6a01177	2.90	5.19	9.22
6a01169	1.33	1.86	2.82

This analysis indicated that during a 0.5% AEP event in 2070 the overtopping discharge would increase for all frontages. The consequences of these increases indicate:

- the threshold for property flooding is likely to be exceeded in wider beach areas; and
- failure to close the flood gates at the slipway may result in structural damage to flood defences.

The analysis of data during a 0.1% AEP event indicated that the overtopping discharge rates would increase in both 2070 and 2115, resulting in significant changes to the flood risk in the area, including:

- failure to close the flood gates at the slipway would result in exceedance of the threshold for structural design by 2070;

- exceedance of the threshold for structural design at the slipway by 2115, even with closure of the gates prior to extreme events; and
- exceedance of the threshold for structural design of flood defences in narrow beach areas by 2070.

Cross-shore Profile Change

During the development of the BMP, it was expressed that the beach geomorphology could potentially alter the amount of overtopping of the seawall taking place during storms, thereby causing a potential reduction in the standard of protection provided by the existing flood defence. Wave action during extreme events can lead to roll-back of the gravel beach, leading to accumulation of beach material at the seawall, and causing an effective ramp for wave run-up and overtopping.

To address this, new analysis was completed specifically for the BMP using the Shingle-B model to determine how the beach could behave under storm conditions if there was more volume on the beach/the beach crest was wider. This section describes the analysis completed, and what measures may be required to avoid the potential issue of run-up and overtopping caused by a higher beach. However, these results are not intended as final design profiles. Water level is very important for determining crest cut-back of shingle beaches and so sea level rise, which has not been considered here, should be considered in any detailed designs along with a wider range of input conditions. Full details of the analysis are presented in a technical note (refer to Appendix A).

5.1 Methodology

Shingle-B is a parametric model for predicting shingle beach profile response to wave action, that generates a dynamic equilibrium profile. The Shingle-B model was used to examine the response of the Seaton beach at locations with (i) a narrow beach width; and (ii) a wide beach width (represented by Profiles 6a01177 and 6a01169 respectively and shown in Figure 5-1) during a 0.5% AEP (1 in 200-year) event.

Figure 5-1 Map showing profile locations assessed on Seaton beach

It should be noted that the model defines an output based on the beach cross-sectional area, water level and incident wave conditions but not the shape of the initial beach profile. This is because studies have shown that varying the beach slope while maintaining the area affects the mode of formation but not the final shape and location of beach profiles (HR Wallingford, 1990 & 2016). The model was recently validated for a similar beach at West Bay, where it effectively reproduced crest cut-back.

5.1.1 Wave Data

Testing was confined to 0.5% AEP events to ensure that the actual standard of protection provided by the defences does not fall below the design standard due to a change in the fronting beach profile. The wave and water level conditions for the 0.5% AEP event were based on the wave conditions which resulted in the worst-case present day overtopping event within the overtopping analysis (CH2M, 2015b), described in Table 28.

Table 28 Input condition for all Shingle-B tests

Joint return period	Significant wave period H_{m0} (m)	Peak wave period T_p (s)	Still water level (m OD)
200	2.4	9.9	3.1

5.1.2 Cross-shore Profile Data

Beach volumes and cross-shore profiles fluctuate over time as the effect of wave action build beach levels or draws material offshore. The most recent survey data obtained from the Southwest Regional Coastal Monitoring Programme, indicated that the most recent survey data (recorded on 12/04/2017) are representative of the typical profiles recorded over the past decade (Figure 5-2 and Figure 5-3).

Figure 5-2 Change in cross-sectional area of beach at Profile 6a01169

Figure 5-3 Change in cross-sectional area of beach at Profile 6a01177

5.2 Results

Figure 5-4 indicates the initial profile in the narrower beach section (Profile 6a01177) and the predicted dynamic equilibrium profile established from Shingle-B. The predicted profile indicates there is likely an insufficient beach width at present to avoid the roll-back of the beach onto the seawall and the associated creation of a ramp for wave run-up. As there appears to be insufficient beach material to prevent the beach rolling-back onto the flood defences, additional modelling was completed to establish how much material might be required in a beach recharge project. Providing additional material to widen the beach, creates an additional platform for the beach to roll-back into, and the necessary material to create the equilibrium profile away from the seawall. This could reduce the likelihood of the crest rolling back onto the wall.

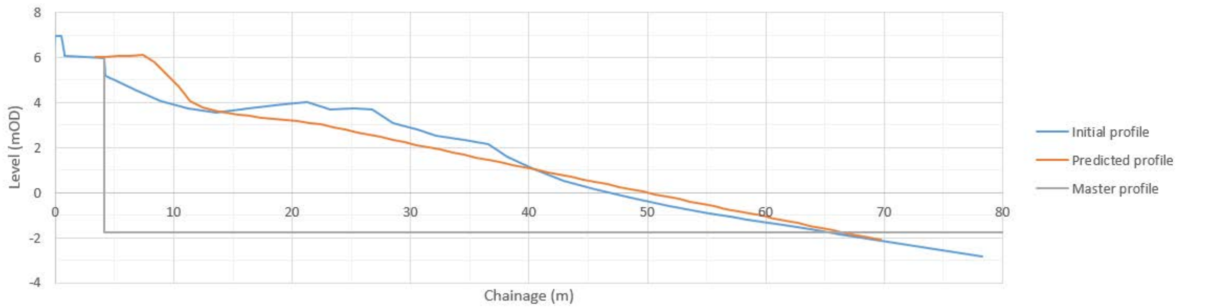


Figure 5-4 Profile 6a01177 exposed to 1 in 200-year event

A simplified design version of the actual profile, with equivalent cross-sectional area, was created to facilitate incremental changes in beach area. The following dimensions were used:

- 1:4 slope between 5.0 mOD adjacent to the seawall to 4 mOD;
- 17 m wide berm;
- 1:5 slope between 4 mOD to 0 mOD; and
- 1:11 slope between 0 mOD down to -3 mOD.

Figure 5-5 provides confirmation that the design profile results in the same predicted profile as before.

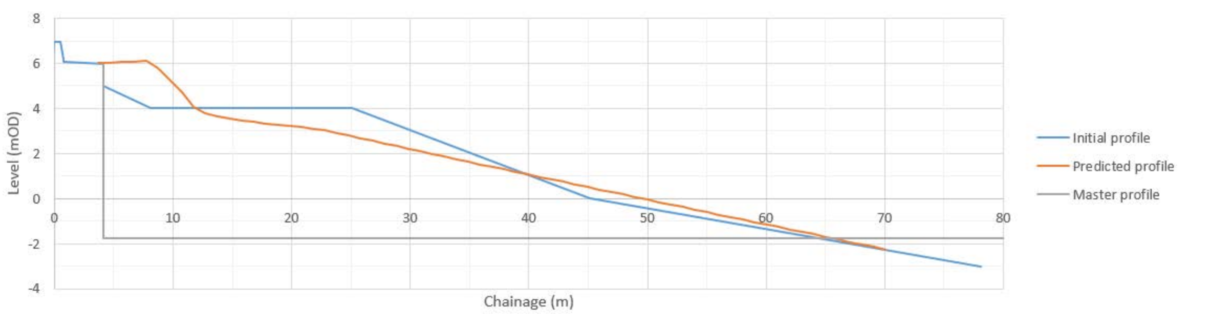


Figure 5-5 Profile 6a01177 Design 1 exposed to 1 in 200-year event

Further modelling was completed to establish the volume of material necessary to avoid beach roll-back onto the flood defences during a 0.5% AEP event in the present day. This was achieved by increasing the width of the central berm to increase the overall beach cross-sectional area (Figure 5-6 to Figure 5-8).

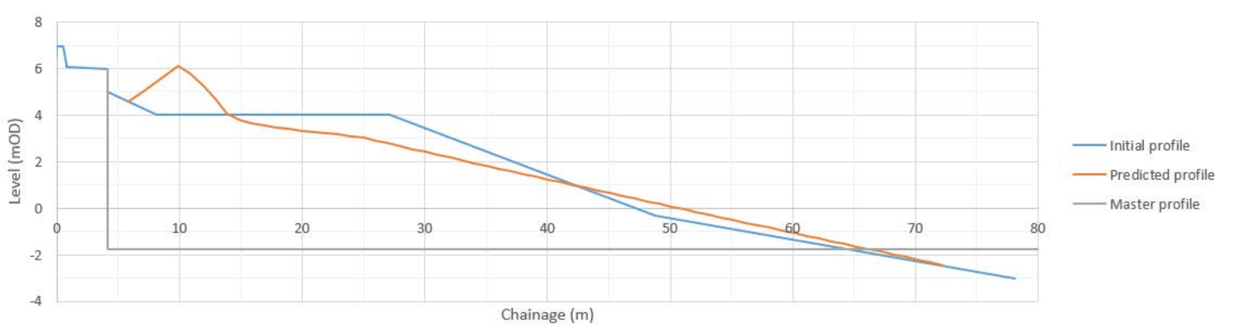


Figure 5-6 Profile 6a01177 Design 2 exposed to 1 in 200-year event

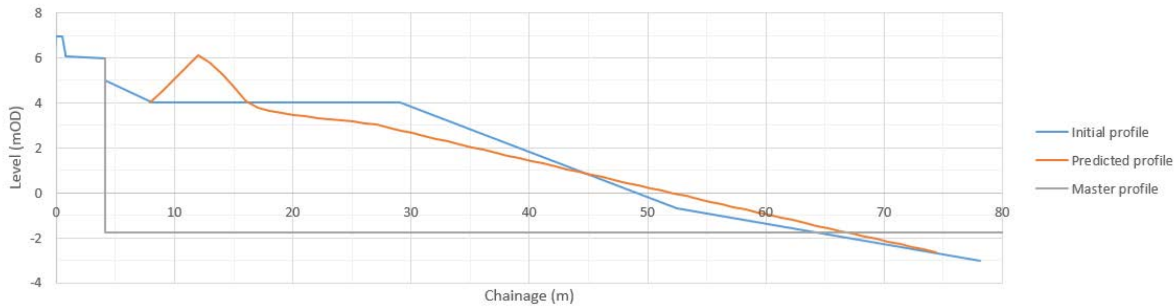


Figure 5-7 Profile 6a01177 Design 3 exposed to 1 in 200-year event

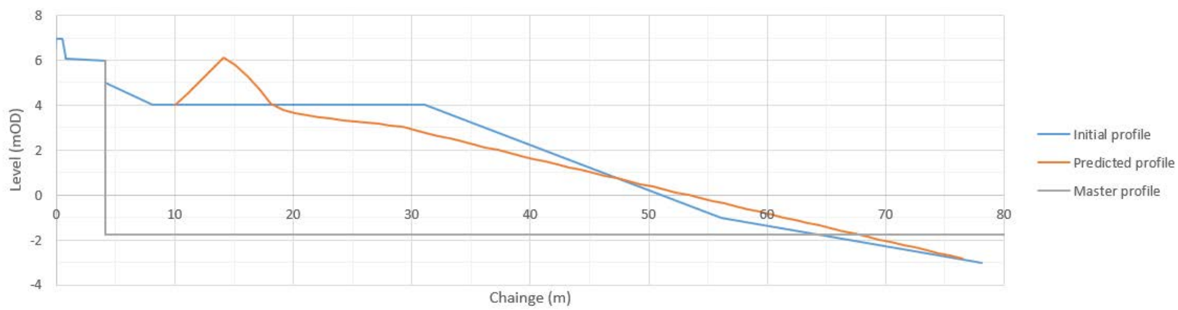


Figure 5-8 Profile 6a01177 Design 4 exposed to 1 in 200-year event

Table 29 summarises the results of the Shingle-B model testing, indicating that the greater the cross-sectional area of the beach, the greater the distance between the seawall and the beach crest of the dynamic equilibrium profile. The landward limit of the profile is predicted with less certainty than the crest position in Shingle-B. Despite this uncertainty, the analysis suggests that an initial crest width of 19m or greater will avoid roll-back of the beach crest onto the seawall for the design conditions.

This specific finding is based on 1:200-year event in the present day. Water level is a very important variable for crest cut-back, therefore different wave conditions will produce different results, and the influence of increasing water levels due to sea level rise could have significant influence for future beach management. Detailed design for beach management should consider a wider range of input conditions.

The comparison of outputs for 6a01177 and Design Profile 1 indicates that reprofiling the beach will have no significant effect on the equilibrium profile.

Table 29 Changes in crest location and cross-sectional area for test profiles

Profile	Area above master profile (m^2)	Increase in area relative to Design Profile 1 (m^2)	Predicted distance between crest and seawall (m)
6a01177	214.9	0.0	3.6
Design Profile 1	214.5	0.0	3.6
Design Profile 2	222.9	8.4	5.8
Design Profile 3	231.9	17.5	7.9
Design Profile 4	241.6	27.1	10.0
6a01169	297.1	82.6	14.3

Figure 5-9 indicates the initial profile in the wider beach section (Profile 6a01169) and the predicted dynamic equilibrium profile established from Shingle-B. The predicated profile indicates there is likely a sufficient beach width in this location to avoid the roll-back of the beach onto the defences and the associated creation of a 'ramp' for wave run-up. The modelled distance between the beach crest of the dynamic equilibrium profile and the seawall was approximately 14m. As such, no further testing has been completed for this profile.

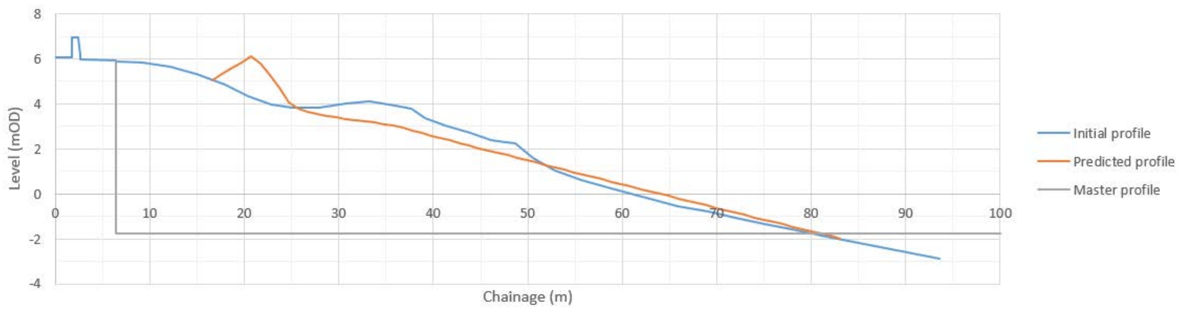


Figure 5-9 Profile 6a01169 exposed to 1 in 200-year event

SECTION 6

Conclusions

This report provides a baseline assessment of the coastal defences between Seaton Hole and the east bank of the River Axe at the river mouth. The baseline assessment included:

- consideration of the historical changes to flood defences and the existing condition of the these structures;
- predictions of wave overtopping discharge for a range of present and future extreme events; and
- modelling of cross-shore profile beach response during extreme wave conditions to assess potential susceptibility to wave run-up.

6.1 Condition of Existing Defence Structures

The condition assessment highlighted several areas where the defences have experienced a significantly reduced performance, and other areas where the flood defences have specific localised issues that might cause wider structural issues if maintenance and repair is not completed.

The locations where the defence structures are in the worst condition are to the west of Seaton, where the structures primarily provide erosion protection at the base of soft cliffs. Even though the defence structures vary in material type and profile, they all appear to have suffered damage to some extent. The long section of rock revetment (Section 3.3) included sections with an inconsistent profile and uneven crest levels. Some sections of the rock revetment were in good condition; however, some locations were damaged or buried by beach material. In these locations, there was also evidence of recent cliff erosion. One further area of significant concern is to the west of Seaton, at the western end of Check House Wall (Section 3.4), where successive defences and defence repairs have been damaged or have failed. The defences here are currently being outflanked. The presence of the rock armour at the base of the cliff contributes helps to reduce the rate of recession of the cliff by dissipating wave energy at the toe and therefore reducing marine toe erosion, however it is not possible to establish how this role is being compromised by a deteriorating defence condition.

The seawall and related structures at Seaton were in generally good condition, with only limited signs of structural damage within the stone block wall and concrete wall. Of more immediate concern was the condition of the flood gates and seals for the gates, which showed clear signs of corrosion and diminished/missing rubber. The inability to close these structures will result in a significant reduction in the standard of protection afforded by the defences. The Environment Agency is currently considering gate modification to improve safety during operation. This will provide also provide an opportunity to deal with structural concerns.

Seaton Spit acts as a flood defence for Axmouth Harbour, although the number of other assets protected by the spit is unclear. The beach area at the spit showed signs of cliffing, both in the upper beach, and in the lower beach area, exposing compacted fine silt material. Exposure of this compacted material increases the potential flood risk in this area by reducing the permeability of the beach, and reducing the ability of the beach to dissipate incident wave energy.

The flood defences in the harbour area and along the eastern bank of the River Axe fluctuates between Good and Fair. The steel sheet pile structures along the eastern bank could not be inspected during the site visit undertaken on 14th June 2017, though they were previously reported to be experiencing ALWC, potentially reducing the overall condition of the structure. Some of the flood defence at the harbour and along the eastern river bank exhibited some signs of damage, including damage to gabion baskets or cracked concrete walls, which should be repaired to ensure existing flood defence standards are maintained.

6.2 Wave Overtopping Conclusions

Previous wave overtopping analysis, conducted as part of Lyme Bay Flood Forecasting Phase 2 project (CH2M, 2015b), was used to understand the flood risk at Seaton. The overtopping analysis was completed in accordance with EurOtop guidance (Environment Agency, 2007), and the analytical method generalised historic beach profiles to allow use of the EurOtop neural network tool. Sensitivity testing was completed as part of the original analysis, concluding that the generalised profiles provide accurate representations of the historic beach profiles.

The analysis indicated that flood risk in the present day for wider beach areas towards the east (described by Profile 6a01169) is a concern for public and vehicle safety under all extreme events. Overtopping rates do not exceed the threshold for property damage (1 l/s/m) until 0.5% AEP and higher events. The threshold for property damage is exceeded when accounting for the effects of climate change in 2070 and 2115, but the threshold for structural damage was not exceeded under any tested condition for the wider beach area.

Analysis of narrow beach areas towards the west (described by Profile 6a01177) indicated more significant flood risk. The threshold for public and vehicle safety was exceeded for all present-day conditions, and property damage is expected to occur for present-day events that are more frequent than 5% AEP. A diminishing standard of protection is expected as the impacts of climate change are realised.

Analysis at the slipway underlines the importance of effective gate closure. Failure to close the gate at the slipway would result in property damage occurring during a 50% AEP event in the present day versus property damage in a 0.5% event if the gate is correctly closed.

6.3 Cross-Shore Profile Change

Examination of the cross-shore profile response using Shingle-B was completed to assess how beaches of varying cross-sectional area could behave under extreme conditions. The analysis suggests that;

- At the western end of the beach, where it is narrower, material may build up against the seawall for the existing profile;
- Material may be prevented from reaching the seawall by extending the width of the beach, however, more tests are required to establish a reliable design profile. The rate of increase in volume of material required to move the crest progressively further offshore gradually increases as the beach slope extends into deeper water;
- Reshaping the beach is not predicted to affect the build-up of material against the seawall for the condition tested. A possible exception to this would occur if the storm duration at a given water level was sufficiently short to prevent the dynamic equilibrium profile from establishing. In these circumstances reprofiling by moving material away from the seawall could offer some benefit; and
- At the eastern end of the beach, where it is wider, material is not predicted to build up against the seawall for the existing profile. Material has been observed to accumulate at the seawall in the past, however the initial beach state prior to the events which led to accumulation at the seawall are not known. Storms occurring when beach levels were depleted may have allowed the beach crest to roll back further. Alternatively, long-shore transport may have affected beach behaviour, and such impacts may be significant at Seaton. Shingle-B does not take into account longshore affects.

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UKCP User Interface. Available online at: http://ukclimateprojections.defra.gov.uk.

Appendix A Shingle-B Modelling Technical Note

TECHNICAL MEMORANDUM

Seaton Beach Cross-shore Modelling in Shingle-B

PREPARED FOR:	East Devon District Council
COPY TO:	Project Team
PREPARED BY:	Jack Eade
DATE:	July 2017
PROJECT NUMBER:	694226
REVISION NO.:	0
APPROVED BY:	Bin Li

The aim of these model runs is to inform the defence baseline for Seaton Beach Management Plan. There is concern that accretion of beach material by the seawall along the promenade increases wave run-up and overtopping which may have contributed towards flooding in the area. It was requested that the effect of changing the beach profile on crest position be investigated to determine if preventing beach build up against the seawall is a viable option for reducing wave overtopping.

These results are not intended as final design profiles. Water level is very important for determining crest cut-back of shingle beaches and so sea level rise, which has not been considered here, should be considered in any detailed designs along with a wider range of input conditions.

Shingle-B

Shingle-B is a parametric model for predicting shingle beach profile response. It is based on an extensive flume study carried out at HR Wallingford in 2014 and was intended as an update to SHINGLE to include the option for users to test bi-modal sea states by defining a percentage of swell in the wave spectrum. Unimodal spectra can be tested by setting the percentage swell to zero. The model will not necessarily return the same results as SHINGLE as different data sets were used to develop the models along with different schematisation of the beach profile. Shingle-B was validated at nearby West Bay where it reproduced crest cut back well. The user manual and model can be accessed online at http://www.channelcoast.org/shingleb/.

Input conditions

Overtopping analysis at Seaton beach was previously carried out by CH2M for the project *Lyme Bay Coastal Flood Forecasting Phase 2* in 2015. Four profile locations were tested two of which were located along the promenade as shown in Figure 1 and were used for this assessment.

Figure 1: Map showing profile locations assessed at Seaton

The previous overtopping assessment considered a number of combinations of wave heights and water levels across a range of joint return periods (JRP) which were transformed nearshore close to the toe of the beach at around -3.0 mOD using SWAN. The condition which lead to the greatest overtopping out of those tested for a JRP of 1 in 200 is shown in Table 1 and was the only input condition used in this analysis.

Shingle-B was calibrated in water depths equivalent to those at most wave buoys in the wave network of the National Network of Regional Coastal Monitoring Programmes (~12 m) which propagate across a fixed bathymetry of dimensions typical along the south coast up to a mobile beach toe. Shingle-B was calibrated for a toe at around -6.0 mOD. A limitation of Shingle-B is that it does not account for more depth limited foreshores. The user manual recommends transforming wave conditions to account for maximum possible wave heights at the toe of the beach in such cases so nearshore conditions were used for these tests.

The most recent surveys (12/04/17) of profiles 6a01177 and 6a01169 were initially tested. Figure 3 shows that the beach cross-sectional area from this survey for 6a01177 is fairly typical and so a simplified design profile (Design Profile 1) was established of equivalent area. This consisted of the following: 1:4 slope from 5.0 mOD next to the seawall down to 4 mOD; 17 m crest; 1:5 slope down to 0 mOD; 1:11 slope down to -3 mOD. The crest was subsequently extended in two metre increments while maintaining all beach slopes for Design Profiles 2-4. No additional tests were carried out for 6a01169 as the beach is sufficiently large to prevent the crest from reaching the seawall under the conditions tested.

Only varying cross-sectional areas were tested as different beach slopes of equivalent area were shown to affect the mode of formation but not the final shape and location of beach profiles in both the Shingle-B (HR Wallingford, 2016) and original SHINGLE (HR Wallingford, 1990) studies.

Results

Figure 5 shows that the existing beach profile at 6a01177 is expected to build up against the seawall for the condition tested. Figure 6 shows that Design Profile 1 replicates this response well. Figure 7 to Figure 9 show the crest of the beach progressively migrating away from the seawall with increasing beach area.

Table 2 and Figure 2 show how increasing the beach area causes the crest to migrate away from the seawall. The relationship is not linear as extending the crest offshore extends the end of the 1:5 slope into progressively deeper water. Beach areas were calculated in SANDS above the master profile which extends from the seawall vertically down to MLWS and then offshore.

Table 2 also shows that the existing area of profile 6a01169 is considerably larger than the largest profile area tested for 6a01177 and as such, Figure 10 shows that the crest is not predicted to reach the seawall under the conditions tested. Figure 4 shows that the current beach area for this profile is relatively low considering the existing trend.

References

HR Wallingford, 1990. *Predicting short term profile response for shingle beaches*

HR Wallingford, 2016. *Modelling shingle beaches in bimodal seas*

Table 1: Input condition tested

Joint Return Period	Spectral wave height, H_{m0} (m)	Wind wave peak period, $T_{p,\ wind}$ (s)	Still water level (mOD)
200	2.4	9.9	3.1

Table 2: Changes in crest position

Profile	Area above master profile (m^2)	Increase in area relative to Design Profile 1 (m^2)	Predicted crest distance from seawall (m)
6a01169_20170412	297.1	n/a	n/a
6a01177_20170412	214.9	n/a	n/a
Design Profile 1	214.5	0.0	3.6
Design Profile 2	222.9	8.4	5.8
Design Profile 3	231.9	17.5	7.9
Design Profile 4	241.6	27.1	10.0

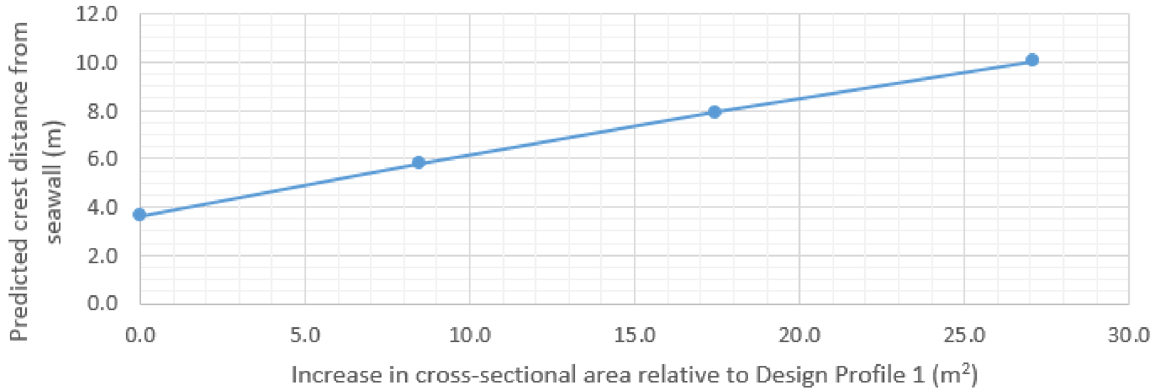

Figure 2: Effect of increasing beach cross-sectional area on predicted crest distance from seawall

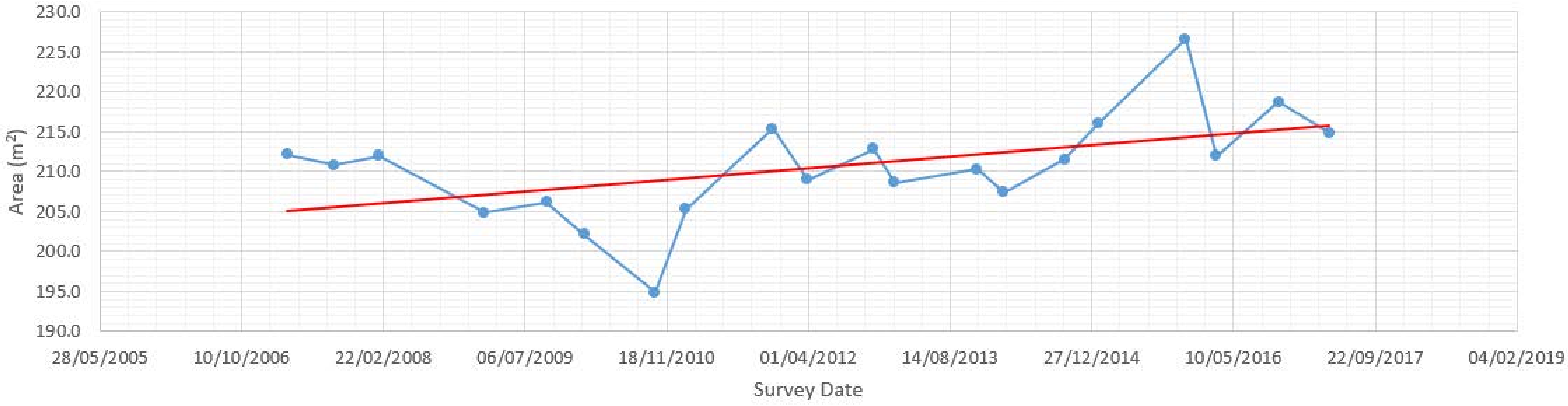

Figure 3: Cross sectional area above master profile for 6a01177

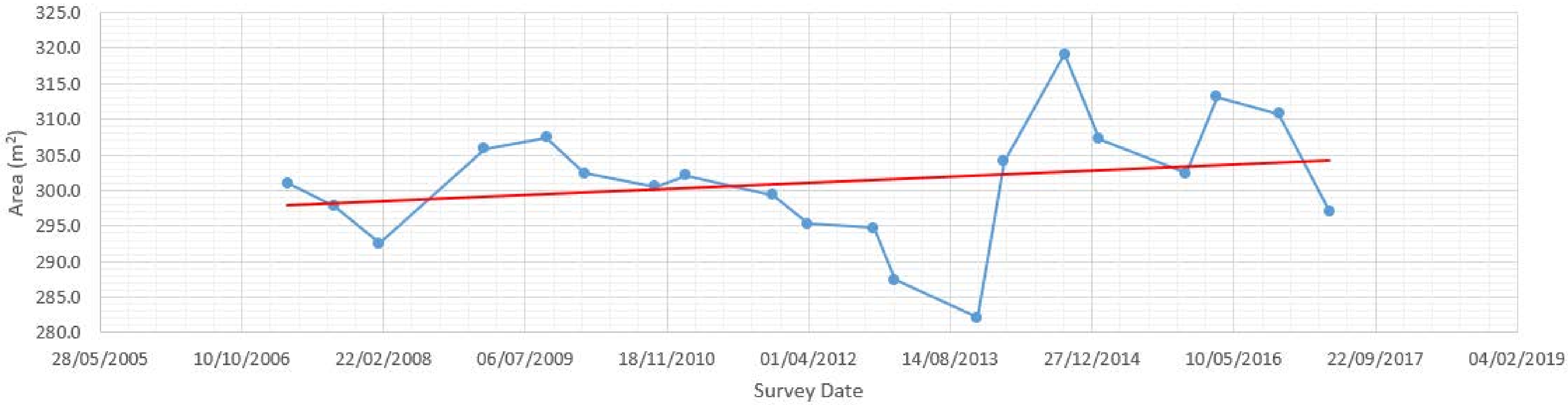

Figure 4: Cross sectional area above master profile for 6a01169

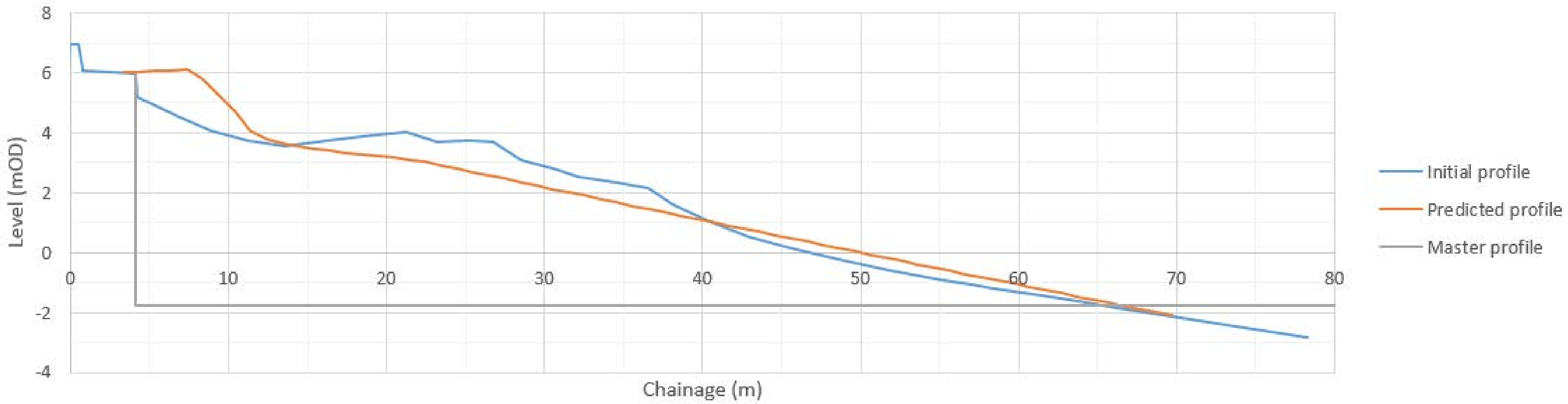

Figure 5: Profile 6a01177_20170412 exposed to 1 in 200 year event

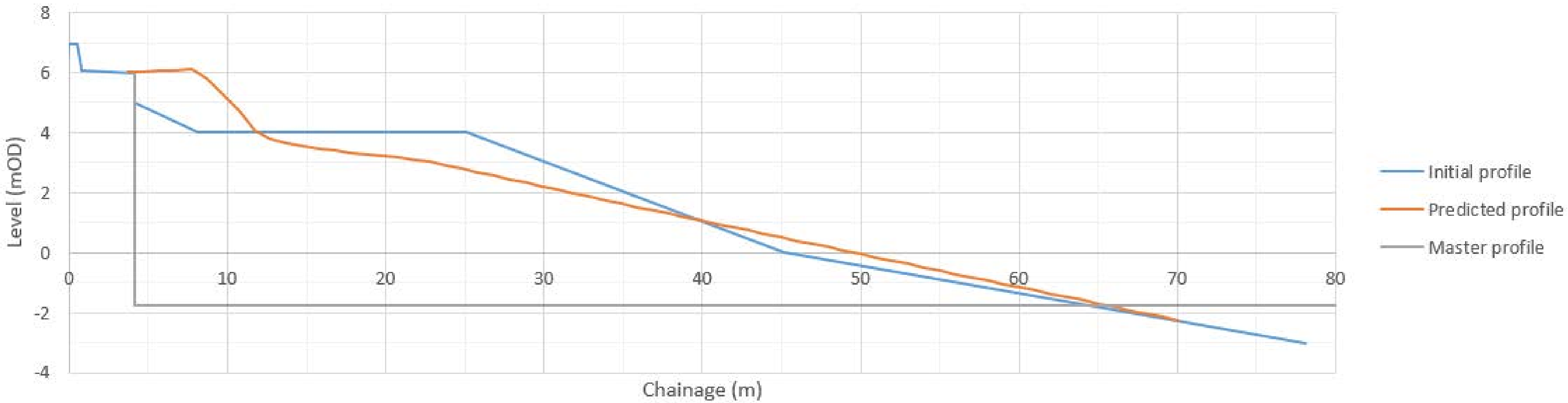

Figure 6: Design Profile 1 exposed to 1 in 200 year event

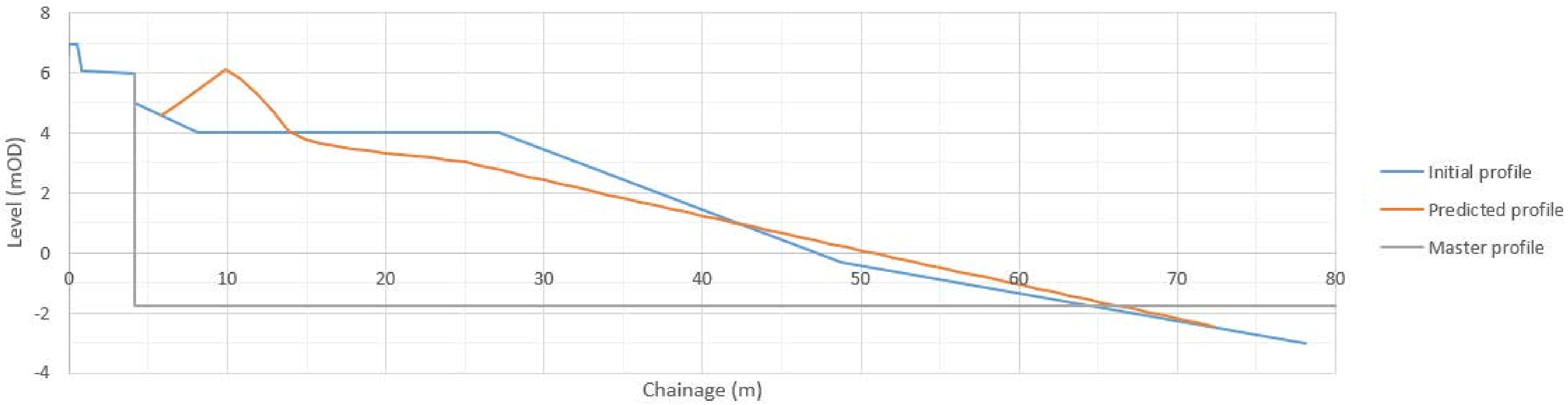

Figure 7: Design Profile 2 exposed to 1 in 200 year event

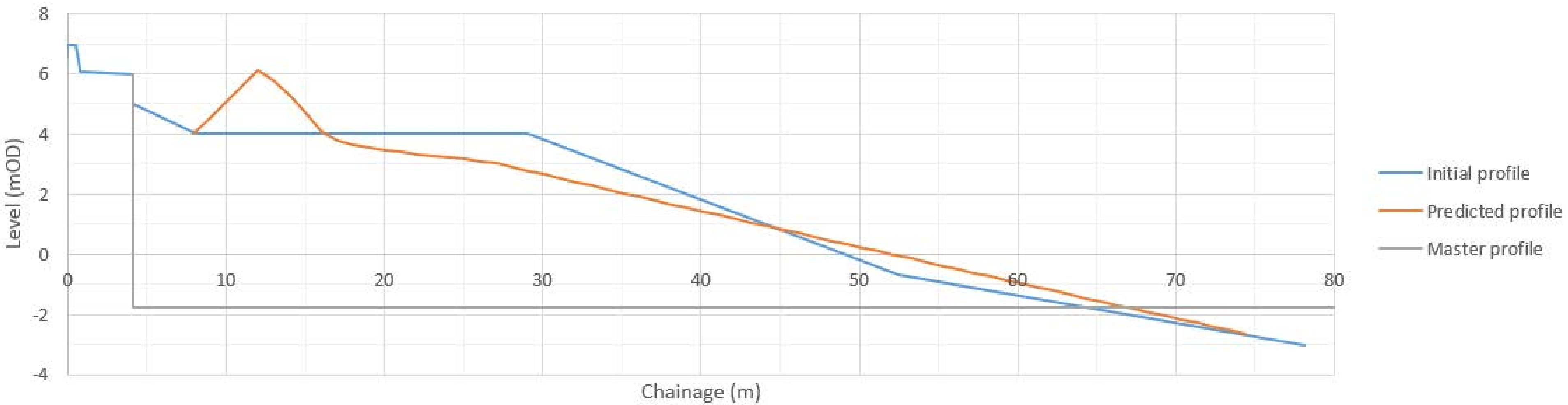

Figure 8: Design Profile 3 exposed to 1 in 200 year event

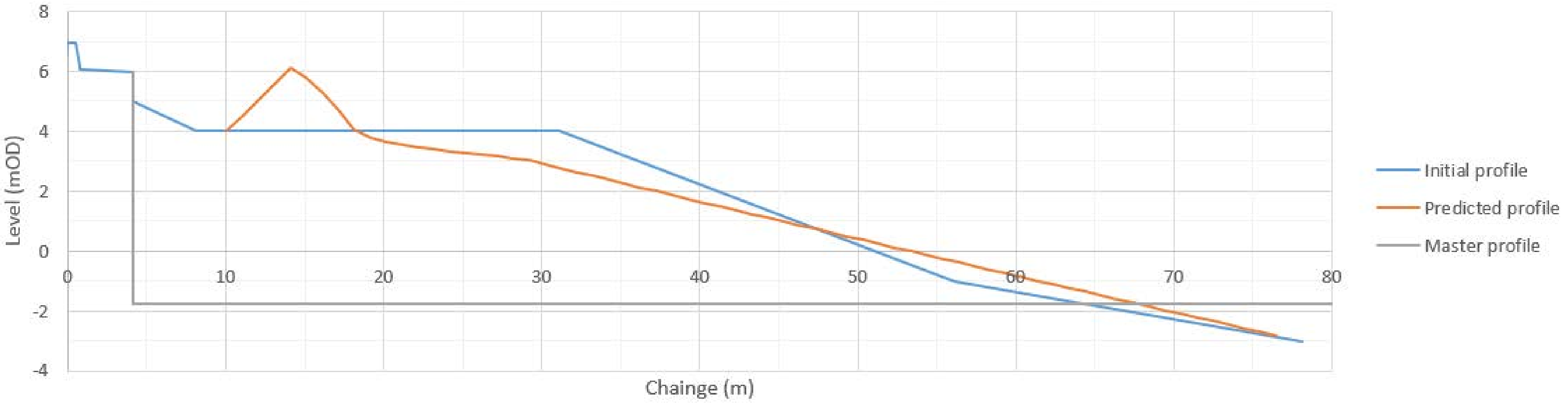

Figure 9: Design Profile 4 exposed to 1 in 200 year event

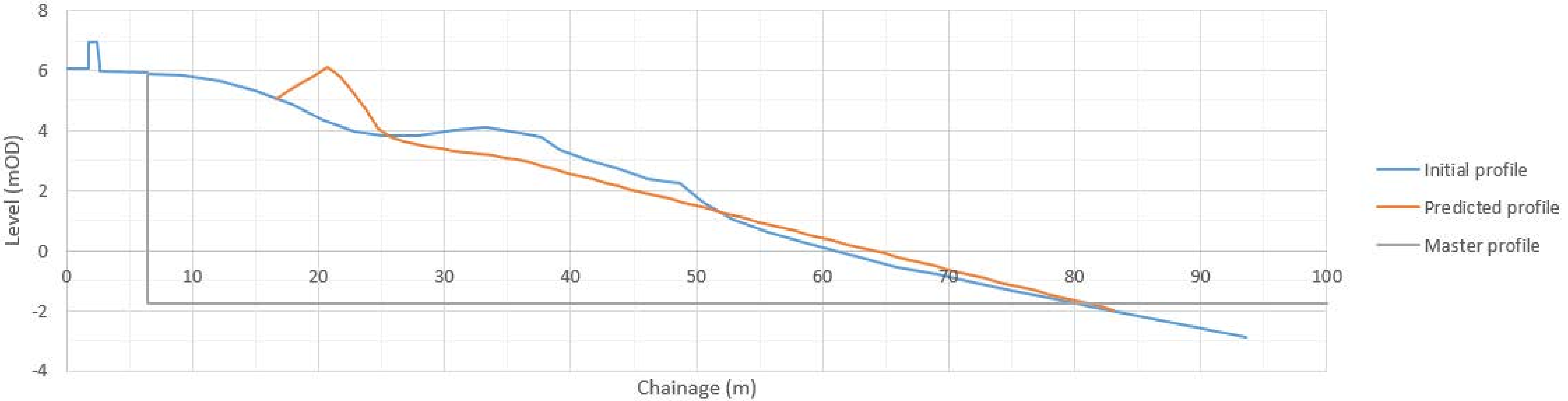

Figure 10: Profile 6a01169_20170412 exposed to 1 in 200 year event