Defence Assessment Report

Sidmouth & East Beach Management Plan

Prepared for

East Devon District Council

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CH2MHILL®

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Document history

Defence Assessment Report

Sidmouth & East Beach Management Plan

East Devon District Council (EDDC)

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Contents

Annexes

1 Introduction

This report provides a baseline assessment of the coastal defences located along the Sidmouth and East Beach Management Plan (BMP) frontage (see Figure 1-1). The purpose of this assessment is to provide information to inform the development of future coastal flood and erosion risk management measures for the frontage as part of the development of an updated and expanded Sidmouth and East BMP. As such, this report includes:

- Information about the history of defences constructed along the frontage, taken from previous studies and reports that have been reviewed as part of this project (Section 2);
- Assessment of the current condition of each element of the coastal defences along the frontage (Section 3);
- Assessment of standard of protection the existing coastal defences provide against wave overtopping and the risk of scour and undermining (Section 4); and
- Conclusions and recommendations for further investigations and possible future management activities relating to the maintenance of the existing coastal defence assets that will inform development of future management options in subsequent stages of the BMP development (Section 5).

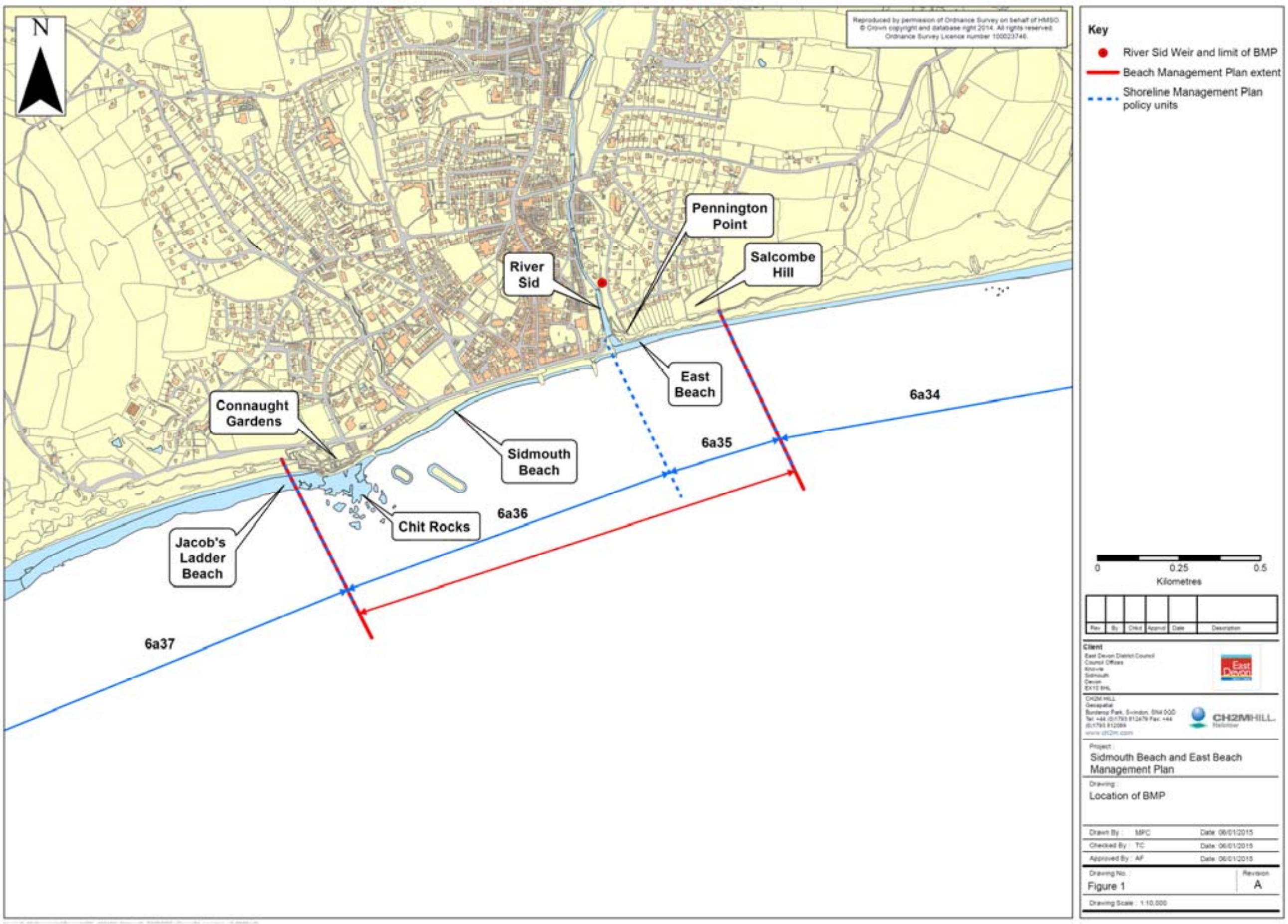


Figure 1-1 Sidmouth & East Beach Management Plan extent

2 Defence History at Sidmouth

2.1 Introduction

The purpose of this section is to outline the coastal defence history of the Sidmouth frontage and provide an account of the main issues which have been identified through past studies. The Sidmouth frontage has had numerous construction phases throughout the past two centuries; the most important known phases are discussed below, and for which reference should be made to Figure 2-1 which identifies many of the key features referred to therein and approximate scheme extents.

Figure 2-1 Key features and approximate scheme extents along the Sidmouth & East Beach Management Plan extent

2.2 Years 1825 – 1875

The following information is taken from Posford Duvivier (1991):

- In November 1824, a great storm occurred which resulted in major erosion of the Chit Rocks, which form a natural breakwater to Jemmett's Point. The effects of the storm prompted construction of several elm timber groynes and breastwork along the length of the Sidmouth frontage between 1825 and 1826. These were the first coast protection measures on the frontage which had previously relied on its natural shingle bank to defend it from the sea.
- In 1835, the first seawall was constructed along Sidmouth Esplanade, extending for a length of 420m. This wall was founded on the shingle bank and not on the bedrock.
- In 1875, Dunning's Pier was built at the eastern end of Sidmouth Esplanade and was later damaged by a storm in April 1922, separating it from the seawall.

2.3 Years 1917 – 1957

The following information is taken from Posford Duvivier (1991):

- Between 1917 and 1919, further repairs and extensions to the 1835 seawall were undertaken.
- In 1920, major works were carried out to construct a lower level seawall between West Pier and the Bedford Steps. This work continued with works at West Pier in 1921.
- In October 1924, the West Pier was destroyed by storm waves and there was severe damage to the defences. The seawall was breached and the promenade was damaged in several locations. Holes were repaired to safeguard the seawall, and in this area the lower level seawall was extended from Bedford Steps to East Pier. The lower seawall extended down to bedrock.
- A replacement structure for Dunning's Pier was built on the same site in 1926, originally called Port Royal groyne and later known as East Pier. To the east of this on the west bank of the River Sid is the Eastern Groyne which was built in 1918, a reconstruction of an earlier training wall. The Eastern Groyne acted as a terminal groyne by reducing the amount of material moving eastwards off the frontage. The groyne also prevented the mouth of the River Sid from migrating west which historically occurred between 1863 and 1879. The defences at Sidmouth can be seen in Figure 2-2 taken in 1947.
- Further repairs to the groyne field shown in Figure 2-2, as well as the Esplanade seawall, were undertaken between 1953 and 1957. The groynes were periodically repaired through to the early 1990's, although for periods the groynes were covered by beach material (Personal Communications, 2015).
- In 1957, a seawall and promenade were built to protect the red sandstone cliffs which front Connaught Leisure Gardens. The seawall is 190m long and was constructed to provide amenity for residents and visitors to the town. The wall extends from Jacobs Ladder beach to Clifton Beach (Posford Duvivier, 1994). Further details are provided in Section 2.7.

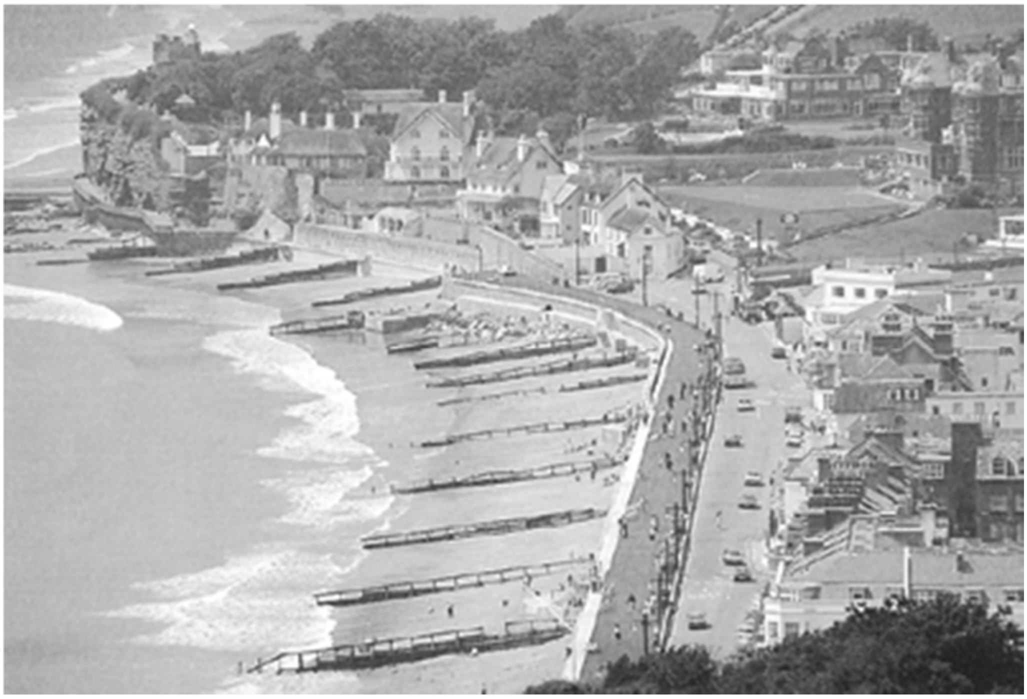


Figure 2-2 Sidmouth frontage and existing coastal defence measures in 1947 (image from: southwestcoastalgroup.org)

2.4 Years 1989 – 1990

In December 1989, the upper masonry wall was breached to the east of Bedford Steps. A number of subsequent breaches in January 1990 occurred to the west of Bedford Steps. Repairs were undertaken as 'emergency works'. The lower parts of the seawall also suffered severe erosion and were showing signs of severe abrasion having occurred over the years. It was apparent that a new scheme was needed to provide the required level of coast protection to the Sidmouth Frontage (Posford Duvivier, 1991).

2.5 Scheme: Phase I (Completed in 1991)

Following a series of storms in 1989 and 1990, a coastal protection scheme was determined necessary to further protect the Sidmouth frontage against the risk of coastal erosion. The scheme was planned to be undertaken in two phases and is detailed in this Section 2.5 and the following Sections 2.6 to 2.10.

Following the series of storms there was extensive damage to the frontage. Substantial volumes of shingle moved to beaches to the east of Sidmouth and were drawn down seaward of the low water mark. The seawall was badly abraded in areas and masonry facing blocks were worn away exposing concrete. Some blocks has also been pulled away. The old wall was exposed in areas where breaches occurred and showed signs of weakness. In some areas, the seawall coping had been lifted. The lowering of the beach, which prior to the storms had been almost to the top of the seawall, also exposed derelict timber groynes. Figure 2-3 shows the Sidmouth frontage following the storm event.

Figure 2-3 Sidmouth beach frontage after the 1990 storms (image from Posford Duvivier, 1990)

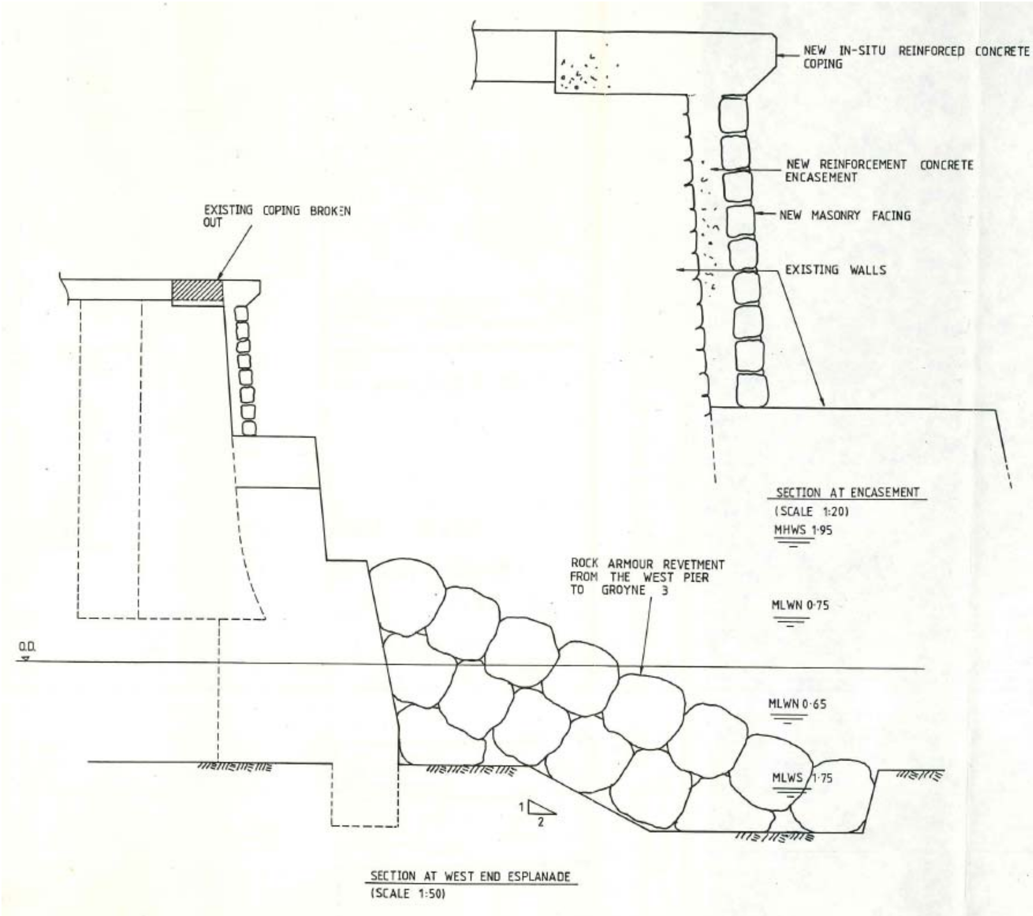
The existing groynes exposed by the lowering of the beach comprising of bullhead railway rails and timber planking were deemed to have reached the end of their working life and were no longer effective due to the combined effects of severe abrasion, marine borers and undermining.

It was advised by Posford Duvivier to undertake a coastal protection scheme in phases. Phase I was to provide urgent measures necessary to safeguard the existing seawall against further damage and comprised the following:

- Encasing the remaining exposed sections of the original masonry seawall
- Encasing the old wall (beach concrete) immediately west of the East Pier

- Providing a low-level rock apron to the sea wall between groynes 1 and 3
- Removing existing timber groynes 1 and 5-12
- Securing the East Pier at its present length
- Encasing the seaward end of the West Pier.

Details of these construction works are shown below in Figure 2-4 and Figure 2-5 showing the proposed seawall repairs/improvements and the groyne removal works respectively.



(Posford Duvivier, 1990)

Figure 2-4 Phase I proposed repairs and improvements to the Esplanade seawall at Sidmouth (from Posford Duvivier, 1990)

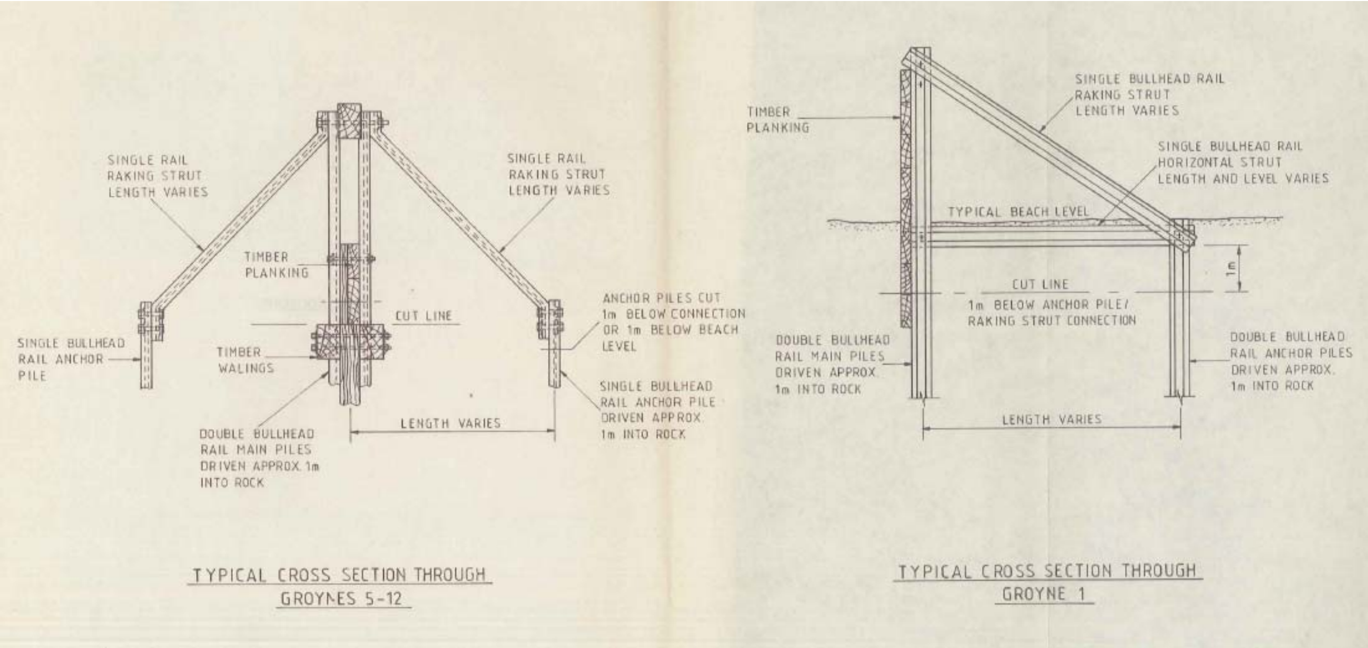


Figure 2-5 Phase I proposed groyne removal works at Sidmouth (from Posford Duvivier, 1992)

2.6 1993 Emergency Works

In January 1993 further lowering of beach levels along the frontage occurred, prompting East Devon District Council (EDDC) to take immediate measures to secure the sea wall from collapse. Posford Duvivier (1993) states that there were outstanding objections to the Phase II scheme (although does not provide details) which would have provided a solution to these issues, so 'Emergency Works' were required to be undertaken instead. The beach lowering can be seen below in Figure 2-6.

Figure 2-6 January 1993 following beach lowering at Sidmouth (from EDDC records)

The works consisted of a low level rock revetment at the foot of the seawall for approximately 400m extending between West Pier and York Steps. Concrete access steps were provided at the existing York, Bedford and Belmont step locations to maintain access for beach users. Repairs to the seawall were also undertaken, mainly pointing to the existing stonework.

A number of options were considered to provide protection to the base of the seawall, such as beach recharge, but due to various constraints the rock revetment was determined to be the preferred solution. The rock revetment would later be incorporated into the Phase II scheme by being buried below the beach recharge level constructed during that phase. This also meant the rock revetment reduced the beach recharge volumes required and thus the corresponding cost of the Phase II works. Details of the rock revetment can be seen below in Figure 2-7. Details on beach recharge levels are provided in Figure 2-10 below.

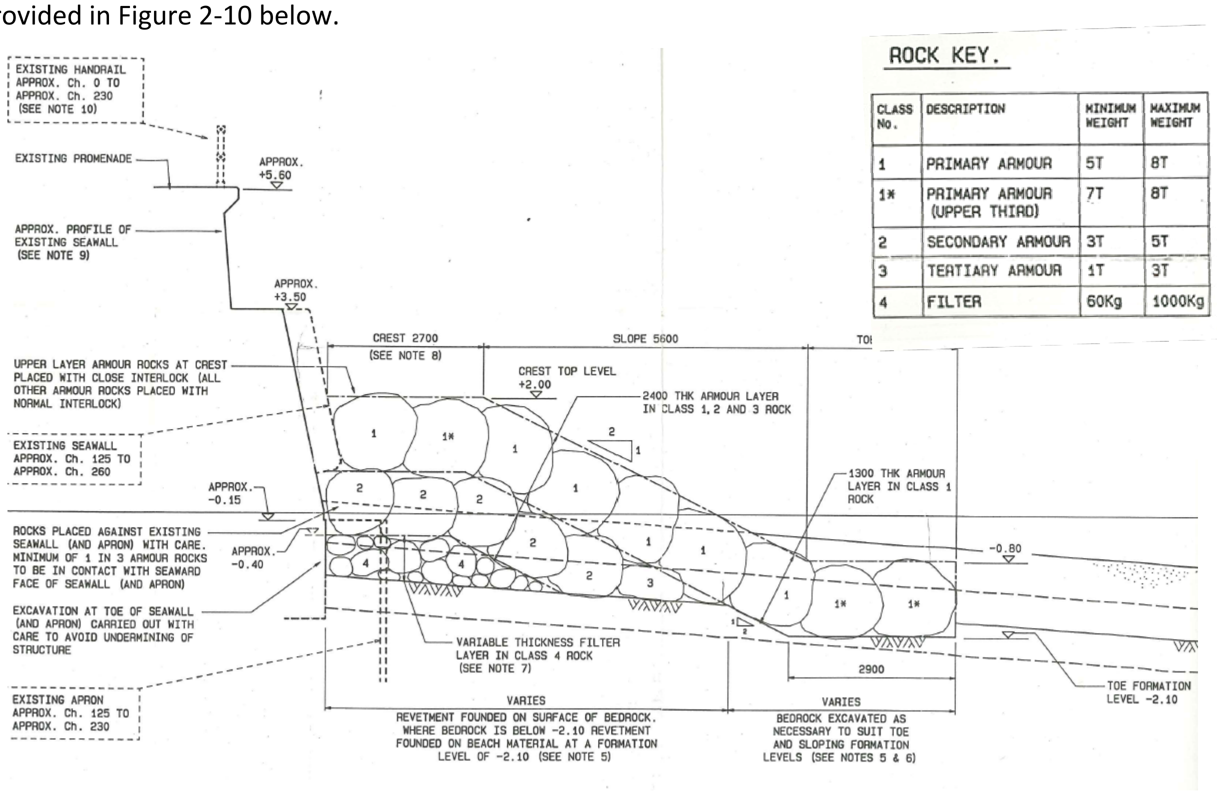


Figure 2-7 Revetment Details as part of the 1993 Emergency Works at Sidmouth (from Posford Duvivier, 1993)

2.7 Connaught Gardens Coast Protection Scheme: 1994

In January 1994 an inspection was carried out of the Connaught Gardens frontage which revealed that there was insufficient shingle and sand adjacent to the seawall to prevent erosion. Adjacent to the seawall, the rock platform known as Chit Rocks had lowered, exposing the foundations of the wall which consisted of a concrete apron along most of its length. There is evidence to show there was undermining of this apron. Figure 2-8 below shows the 1957 seawall and the erosion observed during the January inspection (Posford Duvivier, 1994).

Figure 2-8 1957 seawall and the encountered erosion during the 1994 January inspection (from Posford Duvivier, 1994)

In 1994, a rock revetment was constructed in front of the 1957 seawall extending from the western end of the Phase II works (start of Clifton Walkway – see Section 2.8) for 155m to Jacobs Ladder beach. For amenity and technical reasons, a concrete apron was constructed to protect the seawall return into the adjacent bay to the west (Jacobs Ladder Beach return) for approximately 21m to protect the toe of the wall. In addition, stone repointing was undertaken to the existing seawall masonry. Figure 2-9 shows the typical construction detail for the 1957 seawall and the rock revetment (Posford Duvivier, 1994).

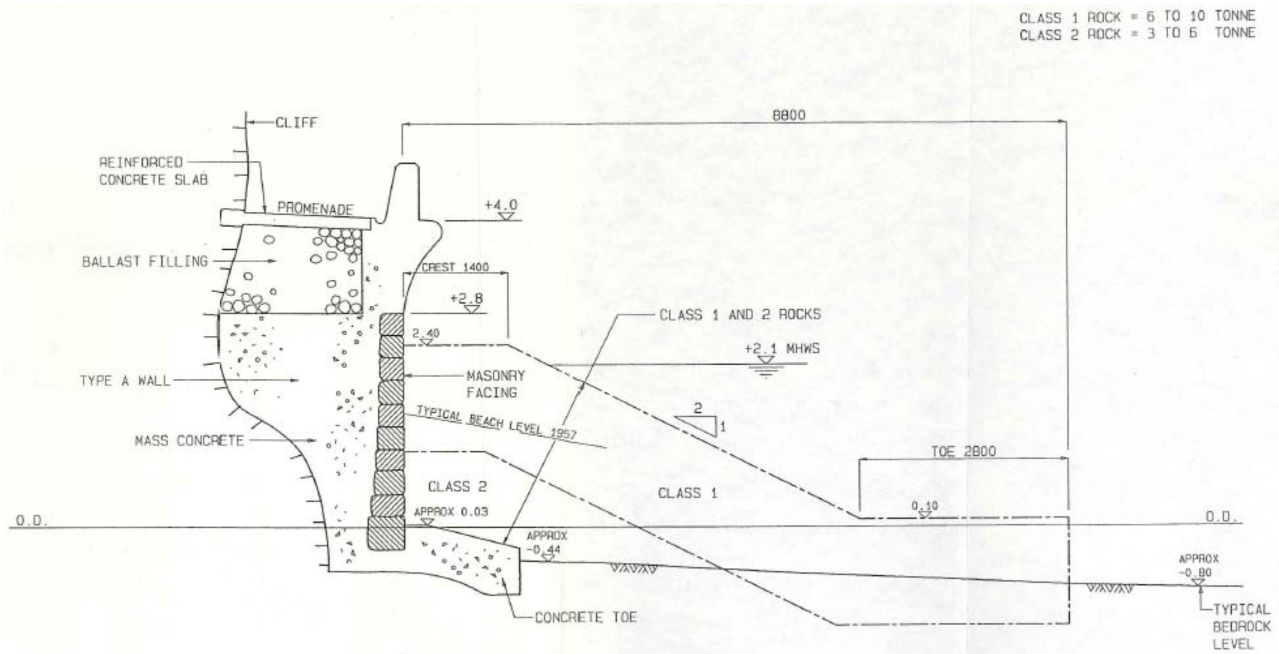


Figure 2-9 1957 seawall and the 1994 rock revetment construction details (from Posford Duvivier, 1994)

2.8 Scheme: Phase II (Completed in 1995)

Phase II of the coast protection scheme at Sidmouth comprised of a range of construction works to further protect the Sidmouth frontage following the 1989/1990 storm events against the risk of coastal erosion over a 50 year design life. The overall rationale for the scheme was to protect Sidmouth seafront without detriment to Salcombe Hill. The view was taken at the time that without any scheme, the Sidmouth seafront and the beaches to the east would all remain at post-1989 lower storm levels. As such, by recharging the Sidmouth seafront with imported material and having structures to hold it in place, then this would be restoring the Sidmouth seafront to pre-1989 beach levels with minimal impact on Salcombe Hill (as there would not be any other sediment entering the system naturally in any case – so recharge would be adding more to the system than nature would otherwise do). With this rationale in mind, the objectives of these works as described in Posford Duvivier (1996) were as follows:

1. To provide long term and sustainable coast protection against coastal erosion.
2. To reduce wave overtopping at the seawall.
3. To restore as far as possible beach levels to that previously enjoyed for amenity purposes.
4. To minimise the visual intrusion of the scheme.

The works were not completed until 1995 as delays had been encountered in acquiring the necessary approvals to undertake the works, though details of the nature of these delays are not given in the available reports (refer to Section 2.6). The works, as described on the as-built drawings (Posford Duvivier, 1995), consisted of the following:

- Promenade re-surfacing and installation of hand railing along the esplanade was undertaken to repair surface defects and improve public health and safety
- Flood gates were installed to span the gaps between the concrete toe wall which existed along the highway side of the promenade
- Constructing a rock revetment at Clifton Beach extending from the 1994 Connaught Gardens coast protection scheme rock revetment through to west pier at the western extent of the esplanade seawall to reduce damage to the masonry seawall
- Removal of Glen Road groyne which was situated between West Pier and Belmont Steps; this structure was ineffective and had fallen into a state of disrepair
- Construction of two large offshore breakwaters to stabilise the beach levels along the Sidmouth frontage and reduce erosion risk which could result in damage to the seawall by reducing wave action at the shoreline (NB: this also has effect of reducing wave overtopping of part of the seawall sheltered by the breakwaters).
- Reinforced concrete encasement of the seawall between east pier and the river training wall (including encasement, but not extension, of the seaward end of the river training wall) as well as coping repair to protect the seawall further and repair the damage resulting from long term abrasion and storm damage
- Construction of three rock groynes at Clifton Beach, East Pier and York Steps to maintain beach levels and reduce the effects of longshore drift. Both York Steps and East Pier groynes included access ramps to the East and West and Clifton Beach groyne included access steps.
- Beach recharge extending between the 1957 seawall and the proposed East Pier groyne for amenity purposes and to further the protection to the frontage against risk of erosion and wave overtopping of the seawall which could otherwise cause it to suffer structural failure as happened in the 1989/1990 storms. Without the recharge, the beach material recovery had previously shown to be a long term process. The beach recharge would bury the revetment constructed during the 1993 Emergency Works. It should be noted that it was anticipated in the design of the scheme that regular recycling of sediment along the frontage, and periodic further beach recharge would likely be required, the need for which was to be guided by ongoing monitoring and frequent review of the beach management plan produced as part of this scheme (Posford Duvivier, 1996), however such works have not been deemed necessary since the scheme (as documented in annual BMP reports produced to 2005 by Posford Duvivier/Royal Haskoning.

Construction details of the works described above are shown in Figure 2-10 to 2-14.

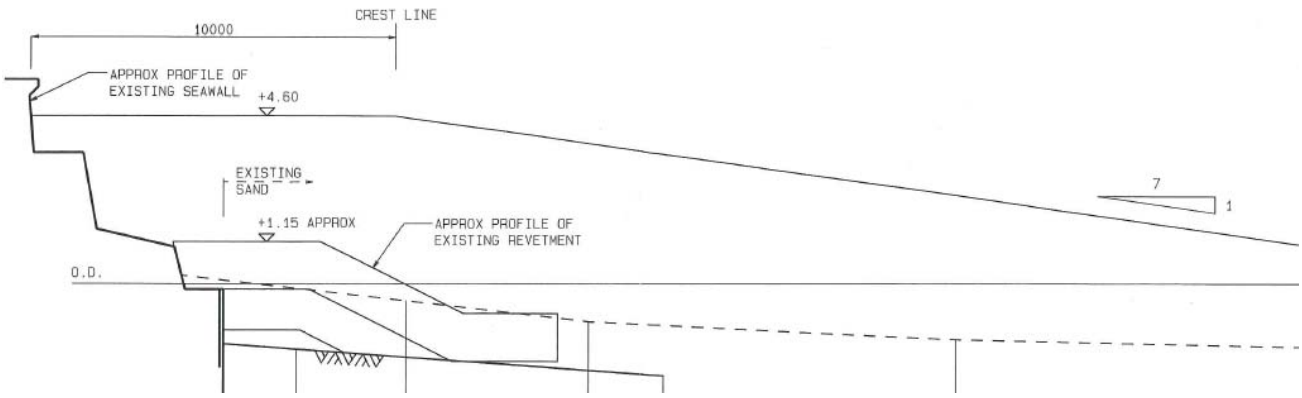


Figure 2-10 1993 rock revetment and beach recharge construction detail for the 1995 Phase II works (from Posford Duvivier, 1995)

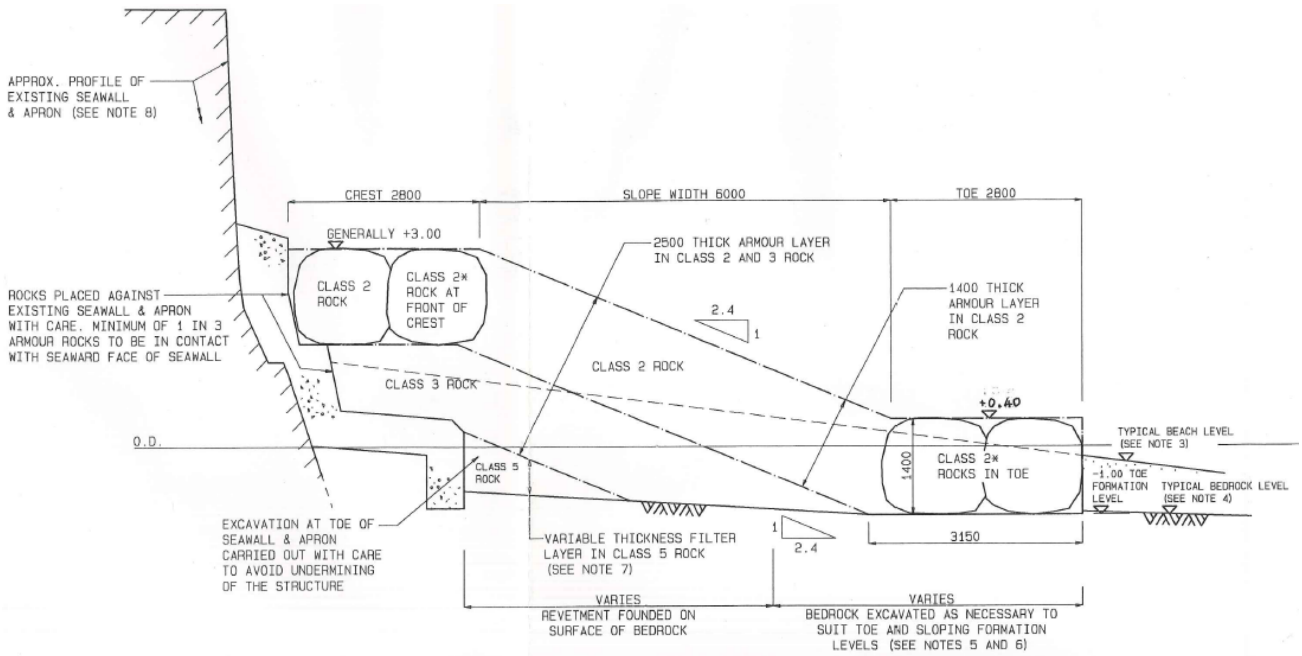


Figure 2-11 Clifton Beach rock revetment construction detail for the 1995 Phase II works (from Posford Duvivier, 1995)

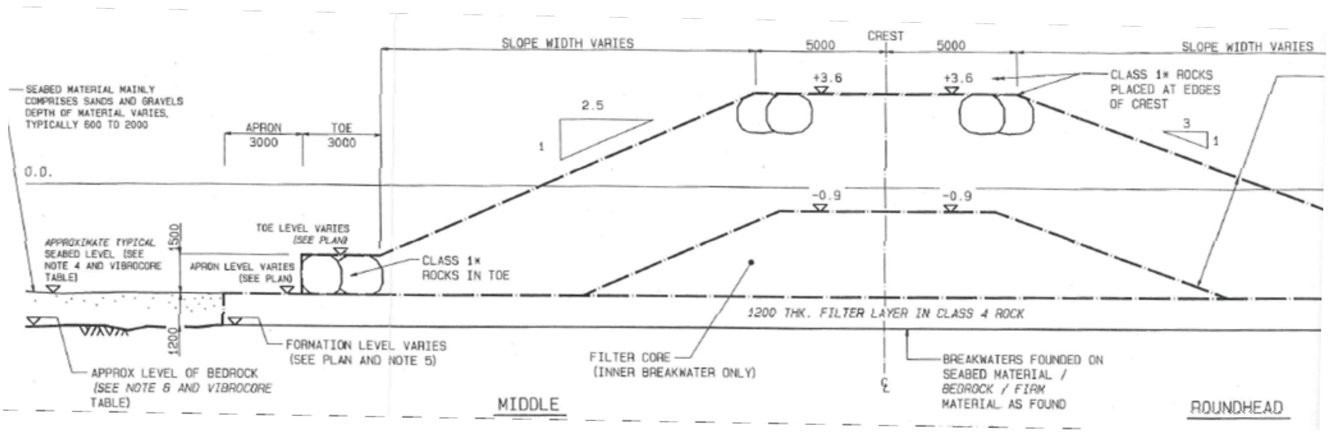


Figure 2-12 Offshore breakwaters construction detail for the 1995 Phase II works (from Posford Duvivier, 1995)

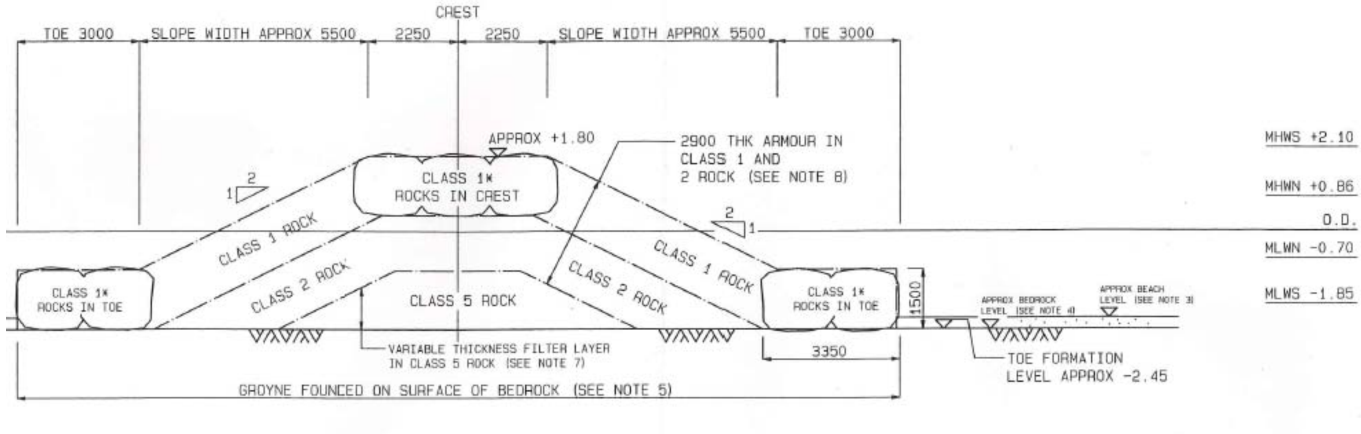


Figure 2-13 York Steps rock groyne construction detail for the 1995 Phase II works (from Posford Duvivier, 1995)

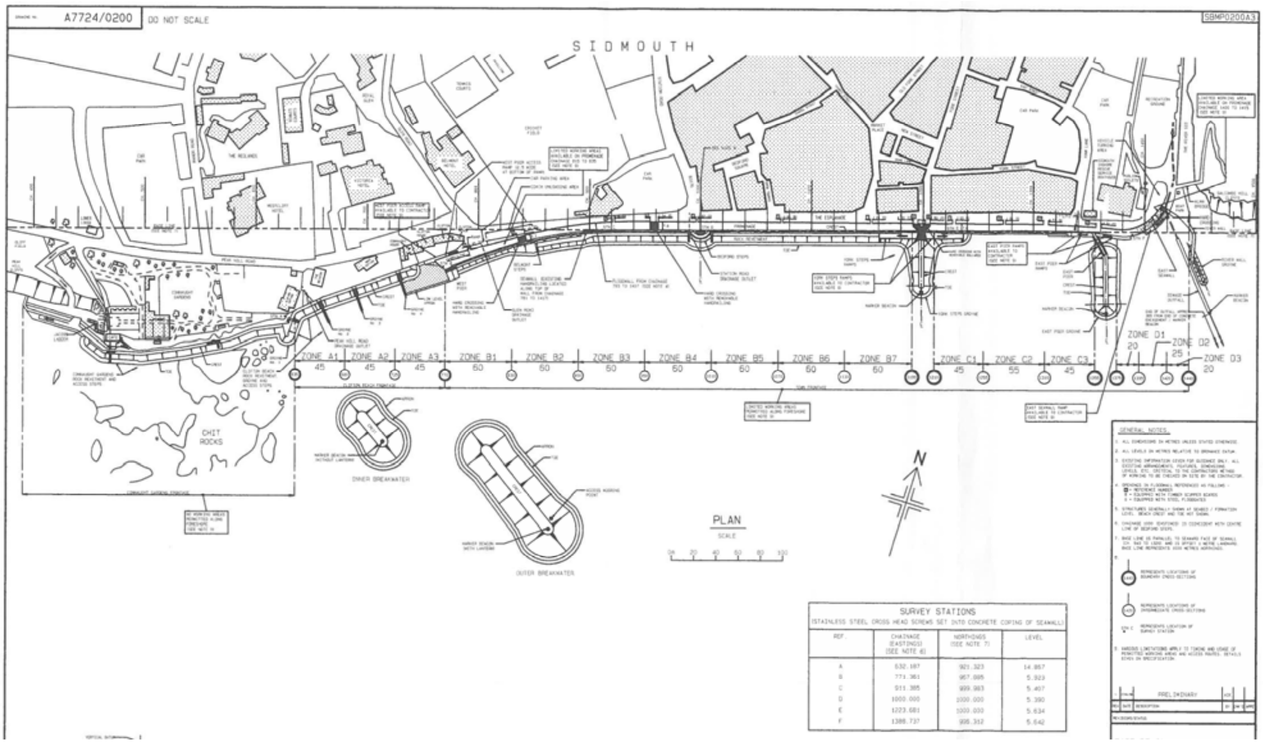


Figure 2-14 Plan of 1995 Phase II works layout (from Posford Duvivier, 1996)

2.9 Clifton Walkway: 1999

Due to the construction of the revetment extending from the Eastern extent of the 1957 seawall to the Esplanade seawall, resident and visitor access was now limited to the beach frontage in front of the revetment. This meant there were access restrictions depending on tide level.

In 1999, the construction of a walkway on top of the rock revetment was undertaken to provide connectivity from the 1957 seawall to the Esplanade. The walkway is a reinforced concrete slab with block paving pinned to the seawall on one side and supported by steel tubular piles the other side, such that if the rock revetment was undermined the walkway would remain stable. Figure 2-15 shows the construction details for Clifton Walkway.

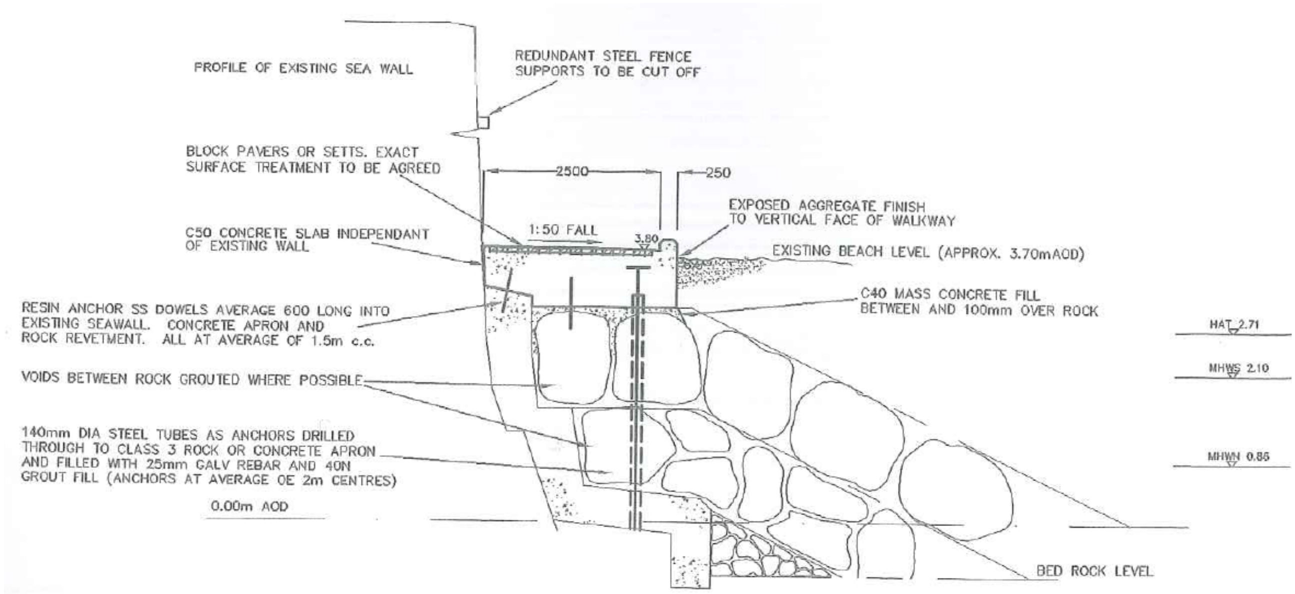


Figure 2-15 Drawing showing the construction details for Clifton Walkway 1999 (from Russell Corney, 2014)

2.10 Scheme: Phase III (Completed in 2000)

Following completion of the Phase I and II works (refer to Section 2.5 and Section 2.8 respectively), the beach showed signs of cut back at the York Steps groyne (Posford Duvivier, 1998) – something that was identified as being possible in the HR Wallingford physical modelling work to the extent that scheme options with a third groyne [now the Bedford groyne) were tested at the time (HR Wallingford, 1993). It was thought the distance between the offshore rock breakwaters and York Steps groyne was too great such that the beach gradually reduced in profile up to the groyne's western flank (Posford Duvivier, 1998). This could lead to scour and undermining of the groyne as well as the reduced beach protection to the Esplanade seawall. The beach levels immediately behind the offshore breakwaters were very healthy such that it could provide a beach recycling source.

In order to address these issues Phase III was commissioned on the basis of options appraisal reported in the 1998 Revised Beach Management Plan (Posford Duvivier, 1998) to undertake the following works to improve the performance of the Phase II scheme:

- Installing a rock groyne at Bedford Steps which matched the construction detail shown in Figure 2-13 for York Steps groyne. This was placed between the offshore breakwaters and the York Steps groyne so that it reduced the distance between control structures and reduced the magnitude of cut back. This was placed at an existing access point and access steps were provided as part of the works.
- Beach recycling was sourced from the beach frontage between the offshore breakwaters and the Belmont Steps and redistributed between Belmont Steps and York Steps groyne.
- Beach recharge and recycling (if material was available) between York Steps groyne and East Pier.

Following completion of the scheme, the performance of the modified scheme with the additional Bedford Groyne was monitored in the same way as Phase II was monitored post-construction to assess the need (or otherwise) for further recycling of sediment along the frontage, and/or periodic further beach recharge. However such works have not been deemed necessary since the scheme (as documented in annual BMP reports produced to 2005 by Posford Duvivier/Royal Haskoning.

3 Condition Assessment

3.1 Introduction

On the 13th January 2014, a visual inspection of the Sidmouth and East BMP frontage (refer to Figure 1-1) was undertaken by CH2M HILL's coastal engineers. The findings of this inspection are detailed within this section. The frontage lengths within the BMP extent are best defined by the defence construction history. Therefore, the condition assessment is split into seven discrete frontages as shown in Figure 3-1 (refer also to Figure 2-1 for key features mentioned in this section). Note, no assessment was made of the undefended coastline to the east of the River Sid; this is covered in the coastal processes baseline being prepared separately as part of the development of the BMP.

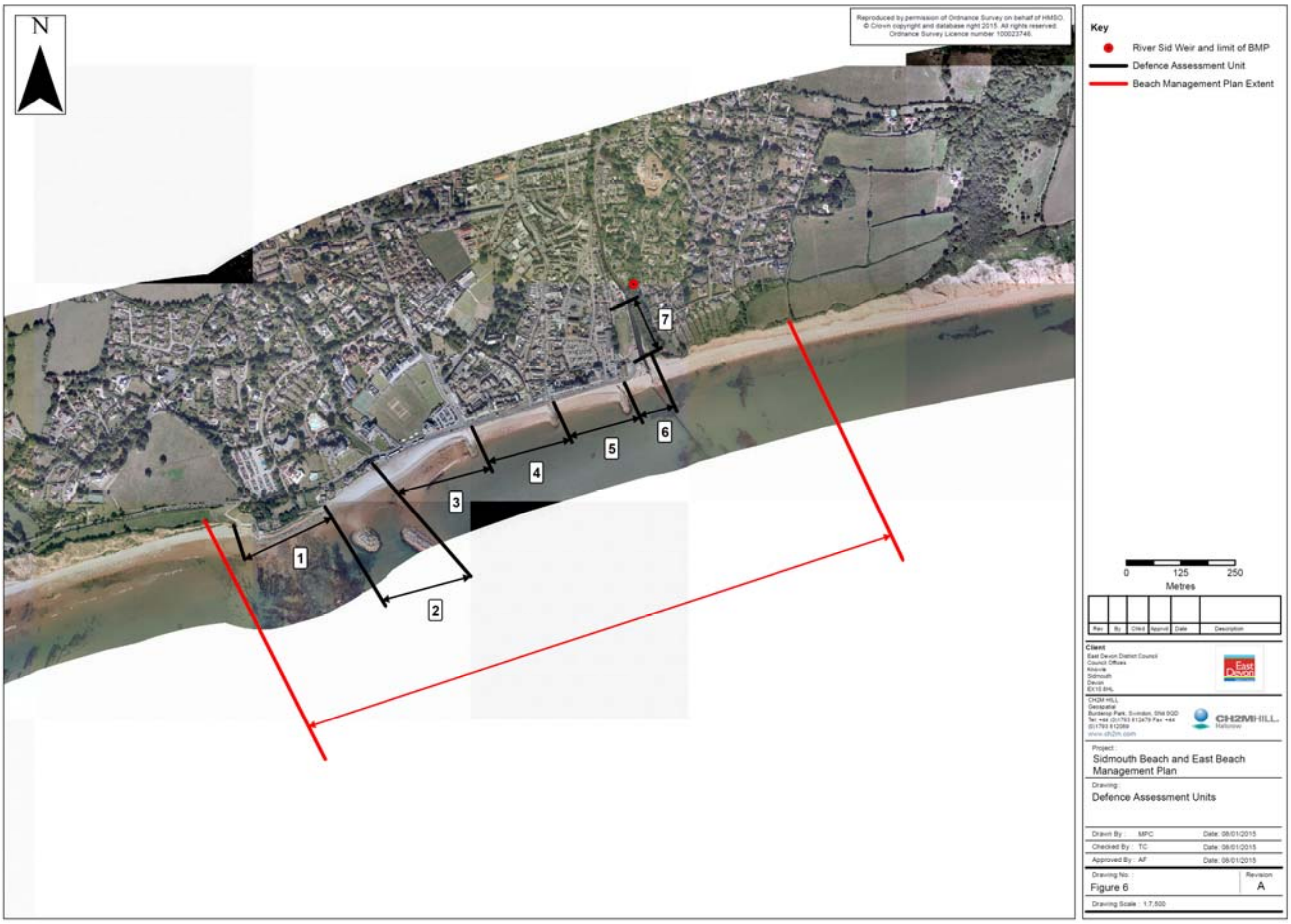


Figure 3-1 Frontage lengths for Sidmouth condition assessment

The frontages shown in Figure 3-1 above are as follows:

1. Jacobs Ladder to Clifton Walkway (Chainage 0 to 200m)
2. Clifton Walkway to West Pier (Chainage 200m to 340m)
3. West Pier to Bedford Steps Groyne (Chainage 340m to 580m)
4. Bedford Steps Groyne to York Steps Groyne (Chainage 340m to 580m)
5. York Steps Groyne to East Pier Groyne (Chainage 580m to 770m)
6. East Pier Groyne to Alma Bridge (Chainage 770m to 940m)
7. River Sid: Alma Bridge to Ham Weir (Chainage 940m to 1020m).

3.2 Methodology

The visual inspection was undertaken in accordance with the Environment Agency's Condition Assessment Manual (CAM). The CAM provides a set of visual indicators in order to assess the integrity and performance of a structure and includes the visible surface defects as well as the asset's surroundings. The indicators allow a condition grade to be determined, of which there are five, ranging from 'very good' to 'very poor'. For each structure type there is a set of visual indicators based on the specific failure mechanisms for the structure and these are outlined in the following sections. In general, the condition grades are based on the descriptions shown in Table 3-1 (Environment Agency, 2006).

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

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

In addition to the CAM, the Environment Agency (EA) has released guidance on relating the condition grades from the CAM to residual life of various structures. This guidance has been used to estimate the residual life of the structures inspected at Sidmouth.

3.3 Frontage 1: Jacobs Ladder to Clifton Walkway

This frontage contains a 190m seawall built in 1957, including an access ramp. In 1994 a rock revetment was added in front of the seawall to protect it from erosion. A rock groyne was added to the end of this revetment as part of the Phase II works in 1995. The condition of these structures is assessed below in accordance with the CAM. Figure 3-2 shows the access ramp, seawall and revetment. Further photos are included in Annex A.

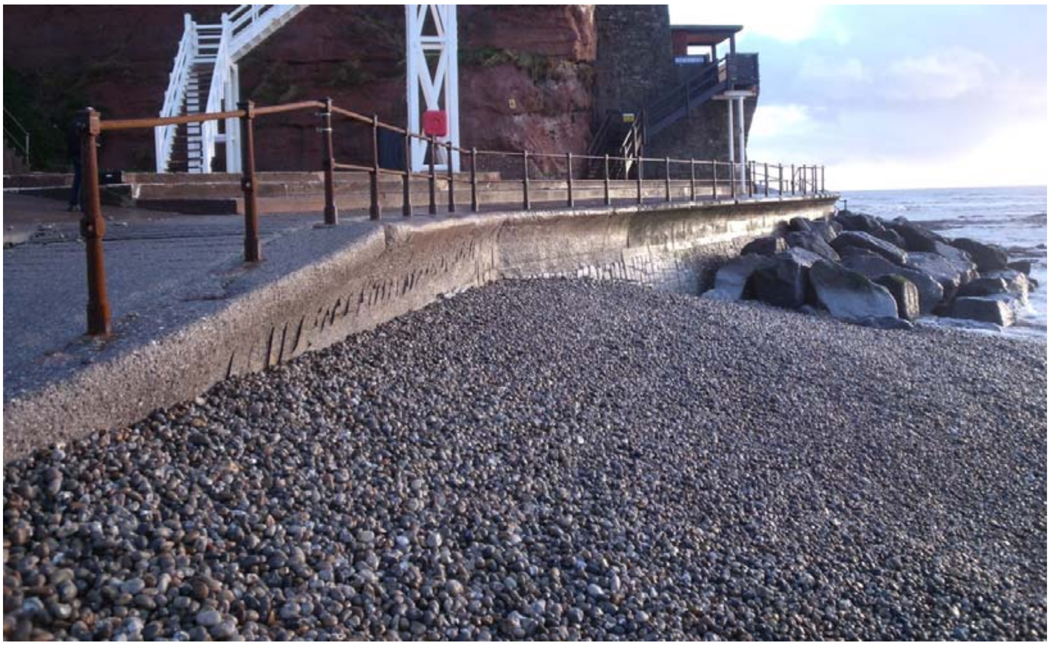


Figure 3-2 Frontage 1 access ramp, seawall and revetment

3.3.1 Concrete Seawall

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a seawall's condition:

- Undermining and scour
- Spalling of concrete
- Heave and slump
- Cracking
- Damage to joints and loss of sealant
- Backfill washout from behind seawall
- Exposed reinforcement.

Condition Description: The seawall access ramp forms the start of the frontage. This structure was not protected by a rock revetment and has consequently suffered extensively over the years. The concrete ramp is heavily abraded, wave attrition has resulted in exposed reinforcement, damaged coping and concrete spalling evident.

Signs of scour were present on the seawall promenade with minor holes and pitting in the concrete. Cracking was observed in the crest wall which sits upon the seawall recurve (refer to Figure 2-8). The cracks were seen directly above rectangular promenade drainage outlets in a number of locations. Concrete coping damage had occurred at a couple of locations. Concrete spalling has occurred in the crest wall at expansion joint locations. There was evidence to suggest these had been filled with mortar, which does not allow expansion and may lead to further spalling. Some expansion joints did not contain the original bitumen sealant or a suitable replacement. This was also evident in the promenade slab expansion joints where bitumen sealant was either missing or the joint filled with mortar.

As the seawall is now fronted by the rock revetment, scour and undermining of the seawall was not able to be assessed. As there are no construction details it is unknown whether this seawall was founded on rock. However, the high bedrock level indicated by the presence of Chit Rocks would suggest it is likely to have been.

Due to the revetment's protection, scour and undermining is likely to be of little concern. Furthermore, there were no signs of heave or slump along the full extent of the wall, which would result from undermining at the seawall toe. There was no visible backfill washout. Due to the presence of the revetment, assessing the condition of the face of the wall, such as the masonry joints and blocks was limited.

Condition Grade: 3 (Fair) – Defects that could reduce the performance of the asset.

Residual Life: For the access ramp section of the seawall which is not fronted by a revetment, the best estimate for complete performance failure is 25-30 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 10-15 years. Therefore, we would suggest an immediate structural repair to this wall is necessary to protect the structure from further damage and thus extend it serviceable life as far as possible, preventing the need for complete replacement in 10-15 years. A large storm event could result in a serious defect developing and leading to structural movement.

For the seawall fronted by the revetment, the best estimate for complete performance failure is 45-50 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 25-30 years.

We would suggest this estimation is reasonable providing the rock revetment fronting the seawall remains intact during these timeframes.

General Observations: There were a number of additional observations made which are advised to be reviewed by EDDC, namely:

- The hand railing in a number of locations is corroded, with full thickness loss observed in some areas. There was evidence this has been replaced in places and not in others; replacement is advised for safety reasons.
- A life ring was found partially buried due to it being situated on the beach. The beach level was likely to be higher than usual and pushed up to the bay's corner following the recent storms. However, knowing the dynamic history of the beach frontage, it is suggested this life ring be moved so as it can be easily accessed and used if required.
- Outfalls were seen to be blocked by beach material as they are not flapped. This is compromising their drainage function and they should be cleaned.
- A maintenance operative was observed applying mortar to holes in surfacing and expansion joints without any initial surface preparation. Where expansion joints exist, mortar should not be placed as this prevents the structure from being allowed to thermally expand. Mortar as a material is also unlikely to be particularly durable as it lacks larger aggregate and has a high water-cement ratio. It is recommended that specialist concrete and sealant repairs are undertaken to protect the asset instead.

See photos in Annex A1, Figures A1 to A12.

3.3.2 Rock Revetment

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a revetment's condition:

- Rock angularity
- Rock packing
- Rock displacement
- Void shingle Infilling
- Undermining and scour
- Sliding and bulging
- Settlement
- Exposed geotextile
- Alignment.

Condition Description: The rock revetment inspection showed that the rock angularity was still good indicating the type of rock sourced was suitable for the application. The rock packing was also good in general, however, the rock is very large which makes tight packing very difficult to achieve. There were large gaps between some rocks, particularly at the toe of the structure which may have occurred due to minor rock displacement or poor rock placement during construction. The voids had not become in-filled meaning wave energy dissipation was still in effect.

There was no evidence to suggest crest settlement, bulging or sliding. The alignment is more or less fixed to the structure fronting the seawall. The revetment does not contain a geotextile.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: The best estimate for complete performance failure is 40-50 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset is 30-40 years.

We consider this estimate to be reasonable. However, the revetment is fronting a seawall and the rock grading is very large with Chit Rocks in front providing protection to the structure. This means the residual life may be longer as the environment could be considered favourable compared to other revetments.

See photos in Annex A2, Figures A13 to A14.

3.3.3 Rock Groyne

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a rock groyne's condition:

- Rock angularity
- Rock packing
- Rock displacement
- Void shingle Infilling
- Undermining and scour
- Settlement
- Exposed geotextile
- Alignment.

Condition Description: The Clifton Beach Groyne was inspected and showed large displaced rocks on both the western and eastern flanks. Reviewing aerial imagery, the structure appears to have lost some shape from the designed structure and potentially flattened mildly. The rock is of large grading meaning tight packing is difficult to achieve. This may be the reasoning for rocks displacing (i.e. the rock packing is not of good quality). Minor shingle infilling was observed at the toe of the structure. There was no evidence of undermining or scour to the structure. The structure does not contain a geotextile.

Condition Grade: 3 (Fair) – Defects that could reduce the performance of the asset.

Residual Life: The EA does not provide residual life estimates for rock groynes. This structure will continue performing for many years providing the rock remains intact. To maintain a good standard of performance, the design profile needs to be maintained and the displaced rock placed back into the core structure.

See photos in Annex A3, Figures A15 to A18.

3.4 Frontage 2: Clifton Walkway to West Pier

This frontage contains a rock revetment with a block paved, reinforced concrete walkway above and a masonry seawall which protects a number of properties behind (Figure 3-3). The reinforced concrete walkway, constructed in 1999, sits above a rock revetment installed in 1995 as part of the Phase II works. It is uncertain when the masonry wall was built but it existed in the historical photo shown in Figure 2-1 in 1947 (67 years ago). In addition, within this frontage length there are two offshore breakwaters. Further photos are included in Annex B.

Figure 3-3 Frontage 2 masonry wall, walkway and revetment

3.4.1 Masonry Seawall

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a seawall's condition:

- Undermining and scour
- Displaced/missing blocks
- Heave and slump
- Cracking of blocks
- Damage to joints and loss of mortar
- Backfill washout from behind seawall.

Condition Description: The seawall showed evidence of staining from years of exposure to the marine environment. There is evidence of minor movement of blocks, some are cracked. The joints have mortar missing in places and there was evidence that mortar repairs had been undertaken to fill joint gaps and face damaged blocks. There was no evidence of structural movement and undermining and scour is prevented through the presence of the revetment and walkway.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: For the seawall fronted by the walkway, the best estimate for complete performance failure is 50-60 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 30-40 years, although it should be noted that as the wall is a retaining structure, failure could occur due to movement of the retained material.

General Observations: It is apparent that the upper section of the wall appears to have been added at a later phase, the time period is unknown. This upper section is of better condition but is reliant on the structure below. It is uncertain as to how the structure was built upon the masonry wall below, the quality of the bond between the two and the effect the additional loading on the wall below may have.

See photos in Annex B1, Figures B1 to B4.

3.4.2 Walkway

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a concrete structure's condition:

- Undermining and scour
- Spalling of concrete
- Movement and settlement
- Cracking
- Damage to joints and loss of sealant
- Exposed reinforcement.

Condition Description: The walkway showed no visible defects. The block paving was fully intact with no damaged joints. There were no signs of water pooling on the surface showing the walkway is shedding surface water to the beach as intended and there is no local settlement. The reinforced concrete showed no signs of spalling, cracking, movement, joint damage or exposed reinforcement.

Condition Grade: 1 (Very Good) - Cosmetic defects that will have no effect on performance.

Residual Life: The EA does not provide residual life estimates for concrete walkways. The structure is in very good condition and providing the revetment in front remains intact the walkway will remain serviceable for decades if maintained.

General Observations: An outfall structure was observed which discharges into a channel running below the walkway. The outfall was in good condition and the screen fronting it was not blocked. Debris and shingle had covered a small section of the walkway during the site visit. Gullies were observed on the walkway which were blocked. It is uncertain as to where these gullies discharge. In large rainfall events these may be essential in keeping the walkway draining sufficiently and should be unblocked regularly. The precast concrete drainage channel on the West Pier access ramp was functioning well and had no debris. A small section of groyne installed in 1824 remains projecting just above the beach level; removal of this feature could be considered for health and safety and environmental reasons.

See photos in Annex B2, Figures B5 to B10.

3.4.3 Rock Revetment

The EA's CAM (EA, 2006) visual indicators for a rock revetment are shown in Section 3.3.2.

Condition Description: The revetment fronts Clifton Walkway and is placed on top of a larger rock revetment built as part of the Phase II scheme. This revetment has a smaller rock grading than the other rock structures along the Sidmouth frontage. The structure had minor displacement of rock and shingle infilling at the toe. The alignment remained as expected (in front of the wall with the crest at a similar height to the walkway) and there were no visible signs of sliding or bulging. There were no signs of undermining and generally rock packing was good. The rock is still angular. There was no sign of a geotextile layer. The quality of the rock is uncertain, although the coloration would suggest it to be of different origin to the other rock structures along the frontage.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: The best estimate for complete performance failure is 40-50 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset is 30-40 years.

We believe this estimate to be reasonable. However, there is uncertainty as to the quality of the rock, the construction and the rock size which is of smaller grading than the other rock structures. For these reasons, this may be an optimistic residual life for this structure and a large wave event could result in significant damage to the structure.

See photos in Annex B3, Figures B11 to B14.

3.4.4 Offshore Breakwaters

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a revetment's condition:

- Rock angularity
- Rock packing
- Rock displacement
- Void shingle Infilling
- Undermining and scour
- Settlement
- Exposed geotextile
- Alignment.

Condition Description: The assessment of these structures is limited as access was not possible during low tide on the day of the inspection. The condition assessment was therefore limited to observations from the shore using binoculars and camera. The rock angularity and packing appeared to be good. There was no visible evidence of exposed geotextile, shingle infilling, misalignment or scour. However, scour could only be assessed for the landward ends of each breakwater. There were a few displaced rocks observed on the eastern flank of the inner breakwater. Google Earth indicates there are also a few displaced rocks on the eastern flank of the outer breakwater, as shown in Figure 3-4 below. Figure 3-4 also shows the breakwaters plan shape is good when compared to the Phase II design drawings. This would indicate that no large settlement and flattening of the breakwater has occurred, which would be indicative of a breakwater beginning to progressively fail.

Figure 3-4 Frontage 2 offshore breakwaters showing good plan profile and displaced rocks

Having reviewed the data supplied by EDDC, it has not been possible to locate an as-built survey of the breakwaters or any subsequent surveys. Future assessments and surveys of the breakwaters cannot be compared to a baseline until one is established. With the anticipated future climate change leading to increased sea levels it is highly recommended that a baseline survey of the breakwater's profiles are undertaken and subsequent surveys on a regular basis (every 5 years minimum) to assess their stability. This should be extended to an area outside of the breakwater footprint to identify any undermining or scour issues.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: The EA does not provide residual life estimates for breakwaters. There are uncertainties in the condition of these structures due to a lack of available data. It is recommended the proposed surveys described above are undertaken so a residual life can be suitably estimated.

General Observations: For the inner breakwater, both the warning sign and marker beacon appeared intact. For the outer breakwater, the marker beacon is intact but the warning sign appears damaged and is currently at an angle, potentially due to wave impact at some point in time.

See photos in Annex B4, Figures B15 to B18.

3.5 Frontage 3: West Pier to Bedford Steps Groyne

Frontage 3 extends between West Pier and Bedford Steps groyne. This frontage contains the original masonry Esplanade seawall which has had various maintenance works over the years, most notably the construction of a lower level seawall in front of the existing wall with a down toe constructed into bedrock in 1920. A reinforced concrete coping and masonry facing was installed during the Phase I works. Hand railing was added and promenade improvements made during the Phase II works. The beach has been recharged (Phase II) and recycled (Phase III) and a large outfall through the seawall exists. Bedford Steps rock groyne forms the eastern end of this frontage which was built during Phase III to reduce the distance between the offshore breakwaters and York Steps groyne (which resulted in severe beach profile depletion moving eastwards). Figure 3-5 shows the seawall and outfall along this frontage length. Further photos are included in Annex C.

Figure 3-5 Frontage 3 seawall and outfall and a shingle beach

3.5.1 Seawall

The seawall visual indicators are shown in Section 3.4.1.

Condition Description: The seawall showed few defects with the reinforced concrete coping in very good condition. There were a couple of expansion joint locations with cracking and missing sealant. It is important that these joints receive an appropriate sealant as part of the asset's maintenance regime. The masonry is in good condition and the joints appear in general to be well maintained. The concrete apron installed in the Phase I works was buried in the shingle beach and therefore could not be assessed for its current condition.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: The best estimate for complete performance failure is 75-80 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 60-65 years.

The failure mechanisms for this wall are realistically much more limited than many seawalls. This is because the coping has been replaced and is in very good condition. The structure has a mass concrete apron and erosion toe into bedrock, and is also fronted by a shingle beach and revetment. The structure was underpinned with mass concrete during Phase I. This means that scour and undermining to the seawall has a very small risk. The coping has shown to be resilient to the wave climate and can be repaired if necessary, and the wall itself is heavily protected from wave attack due to the beach levels and revetment.

For the reasons above, it is suggested that the seawall environment is favourable and a slow deterioration curve is more appropriate. The slow estimate for complete performance failure is 130 years in accordance with the asset deterioration guidance (EA, 2009). The slow estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 100 years.

General Observations: The hand railing is in good condition with no notable signs of corrosion. It was apparent the access steps to the east of the outfall were once two-way and have been adapted to be one-way with a mass concrete pour, the joints appeared to be in good condition. Outfalls were all blocked and are in need of clearing regularly to maintain their effectiveness. Ponding of water was observed at the large outfall to a reasonable depth, potentially as a result of recent storm events, and the deep pool of water currently surrounded by loose steep slopes of beach cobbles and gravel may have health and safety implications.

See photos in Annex C1, Figures C1 to C6.

3.5.2 Rock Groyne

The EA's CAM (EA, 2006) visual indicators for a rock revetment are shown in Section 3.3.2.

Condition Description: The Bedford Steps rock groyne showed large voids between armour rocks. It is likely that rock placement was challenging due to the very large grading of rock. It appears that in places, the voids have been 'plugged' with smaller rocks. These small rocks will be unlikely to last in the longer term when subjected to large storm events. This is a concern for the long term stability of the structure. Shingle infilling has occurred in locations, particularly along the toe of the western flank. The structure is not showing any signs of scour (although visibility was limited) or undermining, and the rock had good angularity. There is no geotextile in this structure and it appears appropriately aligned.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: The EA does not provide residual life estimates for rock groynes. This structure will continue performing for many years providing the rock remains intact. The residual life of this structure is uncertain as the large voids could lead to displaced rocks and shingle infilling. The structure may require regular maintenance, placing displaced rocks back into the core structure to maintain effective beach retention. In general this structure is not in as good condition as the York Steps and East Pier rock groynes (later discussed), despite being built five years later.

General Observations: The concrete access steps were in good condition and no notable defects were observed. The additional rock armour protecting the access steps was well packed and in good condition. The marker beacon was intact.

See photos in Annex C2, Figures C7 to C12.

3.6 Frontage 4: Bedford Steps Groyne to York Steps Groyne

Frontage 4 extends between Bedford Steps groyne and York Steps groyne containing the original masonry esplanade seawall with the previously discussed additions of a reinforced concrete coping (Phase I); lower level seawall (thought to be repairs undertaken between 1953 and 1957); revetment (1993 Emergency Works); and beach recharge/recycling (Phases II and III) (Figure 3-6). The York Steps rock groyne was built during the Phase II works and includes access steps either side. Further photos are included in Annex D.

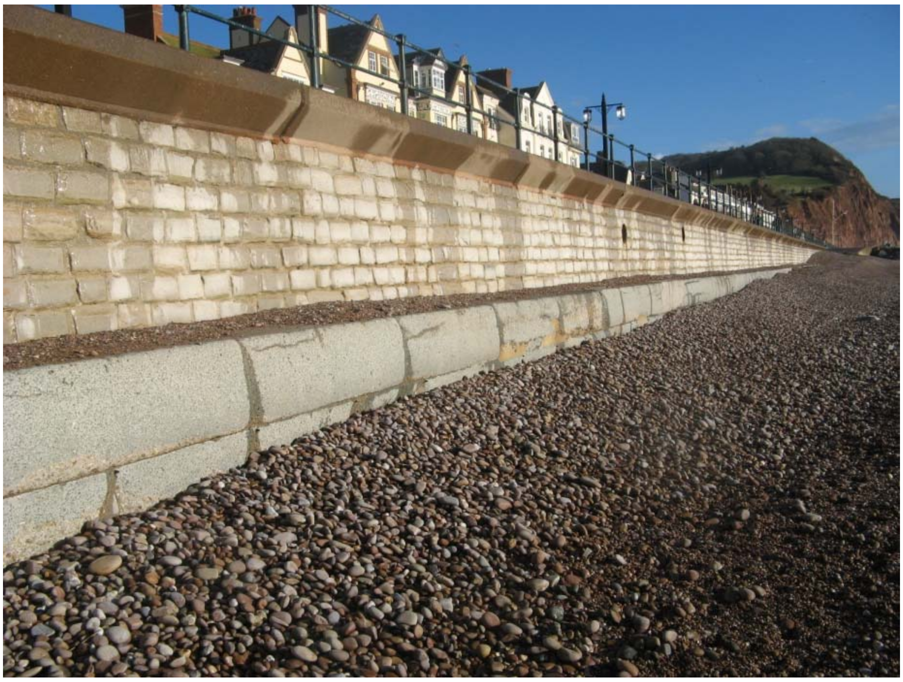


Figure 3-6 Frontage 4 seawall and concrete apron with shingle beach

3.6.1 Seawall

The seawall visual indicators are shown in Section 3.4.1.

Condition Description: The seawall exhibited similar minor defects to that identified in Frontage 3 (Section 3.5.1) with spalling and loss of sealant at expansion joints. Some joints had the remains of a low quality sealant. In some locations it was evident that mortar has been used to fill damaged masonry joints. When this is undertaken, the previous mortar should be completely removed and replaced with an appropriate mortar grade.

The lower level sea wall forming an apron (thought to be mass concrete) along this frontage was exposed on the day of the site inspection. It was observed that this apron has suffered extensive abrasion from wave action and cracks have developed along both the coping and the front face. Mortar repairs were once more observed and, in some locations, there were chunks of mass concrete coping missing. This would indicate that this frontage typically has a lower beach level to the adjacent section and also indicates that the mass concrete used during the Phase I repairs may not be of a high enough grade and not sufficiently resistant to this environment.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: See Section 3.5.1.

General Observations: The hand railing is in good condition with no notable signs of corrosion. Outfalls were all blocked and are in need of cleaning regularly to maintain drainage effectiveness. The revetment placed in front of the concrete apron was exposed approximately half way along this frontage.

See photos in Annex D1, Figures D1 to D6.

3.6.2 Rock Groyne

The EA's CAM (EA, 2006) visual indicators for a rock revetment are shown in Section 3.3.2.

Condition Description: The rock was well packed and had good angularity, with no major voids or notable shingle infilling. There was no visible misalignment, scour or undermining issues and only minor displacement of rock. The structure does not contain a geotextile.

Condition Grade: 1 (Very Good) - Cosmetic defects that will have no effect on performance.

Residual Life: The EA does not provide residual life estimates for rock groynes. This structure will continue performing for many years providing adequate maintenance of the structure, placing displaced rocks back into the core structure and resolving any scour/undermining issues that may develop.

General Observations: The concrete slipways were in good condition and no notable defects were observed. The marker beacon was intact.

See photos in Annex D1, Figures D7 to D10.

3.7 Frontage 5: York Steps Groyne to East Pier Groyne

Frontage 5 extends between York Steps and East pier groyne and includes the esplanade seawall and a rock groyne built during the Phase II works. The seawall has the mass concrete apron and down toe constructed into bedrock (thought to be between 1953 and 1957); reinforced concrete coping and masonry facing (Phase I); and beach recharge/recycling (Phase II). However, the frontage does not include the revetment which was placed in front of the apron along the other frontages. Figure 3-7 shows the York Steps groyne and the esplanade seawall and lower level seawall (apron). Further photos are included in Annex E.

Figure 3-7 Frontage 5 seawall and concrete apron with shingle beach

3.7.1 Seawall

The seawall visual indicators are shown in Section 3.4.1.

Condition Description: The seawall exhibited similar minor defects to that identified in Frontages 3 and 4 (Sections 3.5.1 and 3.6.1) with spalling and loss of sealant at expansion joints. Masonry joints have been repaired with mortar. It is advised that stone re-pointing forms a regular maintenance activity undertaken by an appropriate contractor.

The mass concrete apron along this frontage was exposed on the day of the site inspection. It was observed that this apron has suffered extensive abrasion from wave action and has been severely damaged over the years. Large sections of the apron have been broken out, likely as a result of the combined effects of wave action, thermal cracking and wave impact. This suggests the grading of the mass concrete was not of a high enough quality to resist long term damage in this environment. A repair in the near future is strongly advised as this apron forms the first line of defence from scour and undermining (note this section does not have a rock revetment).

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: The best estimate for complete performance failure is 75-80 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 60-65 years.

The failure mechanisms for this wall are realistically much more limited than many seawalls. This is because the coping has been replaced and is in very good condition. The structure has a mass concrete apron and erosion toe into bedrock, and is also fronted by a shingle beach. The structure was underpinned with mass concrete during Phase I. This means that scour and undermining to the seawall has a small risk, the coping has shown to be resilient to the wave climate and can be repaired if necessary and the wall itself is heavily protected from wave attack when the beach levels are higher. The degradation of the apron should have a high quality repair undertaken to protect the seawall from undermining and scour in the longer term.

A slow deterioration curve may be more appropriate estimate for residual life of this seawall, as demonstrated in Sections 3.5.1 and 3.6.1. However, as the apron is in poor condition and this frontage is not protected by a revetment as well, the best estimate is considered more appropriate for this frontage length.

General Observations: The hand railing is in good condition with no notable signs of corrosion. Outfalls were all blocked and are in need of clearing regularly to maintain their effectiveness. The East Pier slipway had no notable defects and appeared to be in very good condition. Exposed reinforcement was seen protruding from the seawall apron. It is thought that reinforcement may have been used as a support to the apron’s shuttering during construction as the structure is understood to be formed from mass concrete.

See photos in Annex E1, Figures E1 to E6.

3.7.2 Rock Groyne

The EA’s CAM (EA, 2006) visual indicators for a rock revetment are shown in Section 3.3.2.

Condition Description: The rock was well packed and had good angularity, with no major voids or notable shingle infilling. There was no visible misalignment, scour or undermining issues and only minor displacement of rock. The structure does not contain a geotextile.

Condition Grade: 1 (Very Good) - Cosmetic defects that will have no effect on performance.

Residual Life: The EA does not provide residual life estimates for rock groynes. This structure will continue performing for many years, provided adequate maintenance of the structure occurs such as (i) placing displaced rocks back into the core structure and (ii) resolving any scour/undermining issues that may develop.

General Observations: The East Pier groyne was initially a pier formed of concrete blocks including mass concrete access steps. This has since been adapted by placing a mass concrete pour on top to create a “bridge” between the top of the groyne and the seawall promenade. The joints between the mass concrete access steps appear to be in good condition. The concrete block joints are in need of re-pointing, with voids developing in a number of locations.

See photos in Annex E2, Figures E7 to E12.

3.8 Frontage 6: East Pier Groyne to Alma Bridge

Frontage 6 extends between East Pier and Alma Bridge and includes the Esplanade seawall which had replacement reinforced concrete coping and reinforced concrete facing, incorporating an erosion toe, during the Phase II works. This section did not have a lower level seawall built in front during the historical repair works of 1920 and 1953-1957.

The frontage also includes a river training wall which was initially built in 1918 and has been maintained and repaired over the years. The river training wall was extended during the Phase II scheme. Figure 3-8 shows the river training wall on the right and the seawall on the left. Adjacent to the river training wall is an outfall owned by South West Water. The frontage also includes a discrete section of sheet pile wall which runs along the toe of the wall section which extends between the river training wall and the River Sid wall (fronting the River Sid). Further photos are included in Annex F.

Figure 3-8 Frontage 6 seawall (left) and river training wall (right)

3.8.1 Seawall

The seawall visual indicators are shown in Section 3.3.1.

Condition Description: The seawall showed no notable visible defects. There is a reinforced concrete erosion toe to protect this frontage from undermining. The joints are shown as contraction joints when referring to the scheme drawings throughout this frontage length (Posford Duvivier, 1995). However, having reviewed the detail they appear more representative of a standard construction joint. The wall is not at present showing signs of cracking due to thermal expansion/contraction but this should be reviewed as it appears an unusual construction detail.

Condition Grade: 1 (Very Good) - Cosmetic defects that will have no effect on performance.

Residual Life: The best estimate for complete performance failure is 75-80 years in accordance with the asset deterioration guidance (EA, 2009). The best estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 65-70 years.

The failure mechanisms for this wall are much more limited than many seawalls. This is because the coping has been replaced and is in very good condition. The structure has a reinforced concrete facing and erosion toe into bedrock, and is also fronted by a shingle beach. This means that scour and undermining to the seawall has a very small risk. The coping has shown to be resilient to the wave climate and can be repaired if necessary, and the wall itself is heavily protected from wave attack due to the beach levels.

For the reasons above, it is suggested that the seawall environment is favourable and a slow deterioration curve could be more appropriate. The slow estimate for complete performance failure is 150 years in accordance with the asset deterioration guidance (EA, 2009). The slow estimate for a significant reduction in the performance of the asset and defects that would render the structure beyond economic structural repair is 120 years.

General Observations: The hand railing is in good condition with no notable signs of corrosion. Outfalls were all blocked and are in need of cleaning regularly to maintain their drainage effectiveness. The West and East slipways were in very good condition showing no notable defects.

See photos in Annex F1, Figures F1 to F4.

3.8.2 River Training Wall

The seawall visual indicators are shown in Section 3.3.1.

Condition Description: The River training wall has two distinct construction types. The extension, constructed during the Phase II scheme completed in 1995 showed no notable visible defects other than a minor crack in the foundation. It is formed of reinforced concrete constructed in two distinct parts whereby the top section is reinforced precast concrete blocks and the lower section being an insitu concrete foundation, joggle joints were cast to provide interlocking between the two sections. The older construction appears to be a mix of mass concrete and masonry.

The older section of the river training wall showed an exposed manhole ring which appears to have had a temporary repair at some point. In some locations, deep scoured holes have developed and cracking is evident. Spalling, cracking and progressive degradation has begun, with large regions of the wall in poor condition. On the river side of the wall it is evident that ground level is much lower meaning this wall is retaining beach material to a reasonable height. The wall on this face is also severely cracked and damaged and stained. In addition, the wall is suffering scour with large scour holes developing at foundation level on the wall, it is important to note at the toe on this face there is no means of scour protection. Rock has been loosely placed but this is offering little protection from wave damage.

Due to the fact this wall is retaining beach material to a reasonable height and is considerably damaged and scoured at the toe, repairs to this wall are strongly recommended in the short term. Scour protection and temporary repairs to the concrete surface is recommended as a minimum.

Furthermore, video footage and photos during the February 2014 storms showed large waves impacting the river training wall on the beach side. This could be generating large overturning forces on the structure. A large wave event could lead to failure of this structure. These waves were also impacting the river training wall and deflecting up on to the promenade, resulting in large areas of asphalt being removed from the promenade.

Condition Grade: 4 (Poor) - Defects that would significantly reduce the performance of the asset. Further investigation needed.

Residual Life: The best estimate for complete failure of this wall is 15 years and the fastest estimate is 10 years. Limiting the walls exposure to scouring at the toe in the short term will reduce the likelihood of faster failure.

General Observations: The hand railing is in good condition. The hand railing does not continue such that it prevents access along the river training wall and around to the entrance of the slipway, where a considerable height above the beach exists (particularly during periods of reduced beach levels). Hand railing would be advised as a minimum to deter access to the river training wall.

See photos in Annex F2, Figures F5 to F10

3.8.3 Sheet Pile Wall

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a sheet pile wall's condition:

- Lateral movement of pile sections
- Slump and heave behind wall
- Corrosion loss of steel and loss of corrosion protective paint coating
- Gaps and movement at clutches
- Scour and undermining.

Condition Description: The sheet pile wall shows some minor lateral movement of a few piles, however, this may have been during installation, the existing gravel and cobble ground may have led to piles diverting while they were driven. Minor corrosion loss and marine growth can be observed. The piles appear to be cut flush and bolted with a steel capping beam to the mass concrete fill behind the piles, all bolts appear to be intact. It should be noted that during the site visit the river levels meant that no observations could be made on whether the piled wall is suffering scour or undermining. However, there were no signs of serious pile movement which would be expected. In addition, it is expected that these would have been toed into bedrock, which is exposed at a low level further upstream.

Condition Grade: 2 (Good) - Minor defects that will not reduce the overall performance of the asset.

Residual Life: The best estimate for complete failure of this wall is 40 years.

General Observations: At the seaward edge of the sheet pile wall there appears to be scour of the mass concrete infill behind the wall. If this continues, the structural integrity of the wall will deteriorate faster, with corrosion expected on both faces of the pile. It is advised that this scour hole is repaired with a mass concrete pour and steel plate, for example.

See photos in Annex F1, Figures F11 to F14.

3.8.4 South West Water Outfall

The existing outfall is constructed from mass concrete with sheet piled flanks, presumably for scour protection. It is unclear when this structure was built.

The existing outfall was not inspected during the site visit as it was largely inaccessible due to the tide. This outfall was, however, inspected by Hyder during January 2013 who estimated the condition of the outfall within the intertidal zone as Condition Grade 1. The condition of the remaining portion of the structure below MLWS is unknown (Hyder, 2013). This structure does not have a formal function as a coastal defence but will prevent littoral drift, especially when beach levels are lower. Further assessment is required on the outfalls' effect on the coastal processes along the Sidmouth frontage.

3.9 Frontage 7: River Sid: Alma Bridge to Ham Weir

Frontage 7 extends between Alma Bridge and Ham Weir along the River Sid and includes natural cliffs on one side of the bank and what is suspected to be a mass concrete wall with walkway on the other river bank. At the upper extent of the frontage is Ham Weir. Structural improvements to Ham Weir and the River Sid wall were made during the 1963 Sidmouth Improvement Scheme (Devon River Board, 1963). This included raising the heights of the existing flood walls. The assessment of Alma Bridge will be limited to the pier footings. Figure 3-9 shows Alma Bridge and the River Sid river wall in the background.

Figure 3-9 Frontage 7 river wall in the west background and Alma Bridge in the foreground

3.9.1 Alma Bridge

The visual indicators for the bridge piers are those listed in Section 3.3.1 and 3.8.3.

Condition Description: The bridge piers appear to be founded on concrete and protected by sheet piles. It is apparent that a repair to the bridge piers was undertaken recently. Frederick Sherrell undertook an inspection and ground investigation of the bridge piers in 2012. The inspection revealed undercutting of the existing concrete bridge piers. Repair options were provided to prevent further scour to the bridge piers. It is assumed that one of the options was constructed following the report which consisted of sheet piles to surround the bridge pier foundations and a mass concrete pour to fill the annulus. Both the sheet piles and mass concrete are in very good condition and are not showing signs of defects.

Condition Grade: 1 (Very Good) - Cosmetic defects that will have no effect on performance.

Residual Life: Based on the typical design life of concrete and the best estimated residual life of a sheet pile wall, the bridge pier foundations should last a minimum of 50 years.

See photos in Annex G1, Figures G1 to G2.

3.9.2 Mass Concrete River Wall

The EA's CAM (EA, 2006) lists the following visual indicators for assessing a concrete wall's condition:

- Undermining and scour
- Spalling of concrete
- Heave and slump
- Cracking
- Damage to joints and loss of sealant
- Backfill washout from behind seawall.

Condition Description: The existing wall is thought to be a mass concrete wall. There has been an extension in reinforced concrete to the wall of approximately 600mm height which is thought to have been constructed during 1963 as part of the Sidmouth Improvement Scheme (Devon River Board, 1963). Unfortunately, it has not been possible to obtain details of the existing wall on which the extension was built.

The wall is showing signs of severe scour which has led to the construction joints being increasingly exposed. Lines of scoured joints have developed along the walls chainage, typically at three levels. In some places, localised scour holes have developed in the wall, some of which appear to be in the order of 200-300mm deep. Vegetation has grown between the original wall and the reinforced concrete extension. The joint between these two periods of construction appears to be in poor condition and will be subject to deterioration caused by the growing vegetation and freeze thaw, leading to concrete spalling.

The most concerning structural defect is the undermining and scour to this wall. It appears the original mass concrete wall was founded on existing low lying bedrock layer. This layer has become increasingly scoured over more than 50 years which is leading to deep scoured holes beneath the wall. At some point these holes will become continuous lengths where the wall is undermined and localized sections of the wall could begin to fail. It is recommended that some remedial works are undertaken in the short term to stabilise this wall and reduce the risk of scour and undermining.

The Environment Agency's Asset Information Management System records that the above defence is categorised as being in Condition 2, i.e. above average, which is understood to be the 'combined' grading. The defence at that location is a compound structure comprised of the river side wall (mass concrete structure described above), walkway and retaining wall with high ground behind (which forms the Recreation Ground / Play area).

The last T98 inspection, carried out on 12 March 2014 by military colleagues, assessed the defence as having a grading of 3 (Fair) for the mass concrete wall and 2 (Good) for the other structures. This grading is the same as the most recent EA staff inspection completed on 29/08/2012 (Environment Agency, 2015). The primary defence is the mass concrete structure and failure of this asset would lead to fast deterioration of the other assets. The condition grading will therefore be associated with the mass concrete wall.

Condition Grade: 3 (Fair) - Defects that could reduce the performance of the asset.

Residual Life: The best estimate for complete failure of this wall is 15-30 years.

General Observations: The walkway slab along the top of this wall is in good condition and not showing signs of cracking. This suggests the river wall is stable at present. Slump and heave would be expected should the wall begin to fail as a result of scour and undermining. The handrail appears to also be in good condition. This agrees with the condition grading undertaken by the Environment Agency above.

3.10 Additional Observations

In addition to the points noted above in Sections 3.3 to 3.9, the following additional points should be noted with regards the coastal defence assets along the Sidmouth and East BMP frontage:

- As noted in Section 3.1, the condition assessment was undertaken by visual inspection by CH2M HILL's coastal engineers on the 13th January 2014. Following this site visit several extreme storm events occurred in February 2014.

  It has been observed through photography and informal site visits following those storm events, that further damage occurred to the Sidmouth frontage. The most notable damage was to the Jacobs Ladder frontage where some of the existing crest slabs were reportedly lifted and moved out of position due to the wave forcing experienced during the storms. The promenade was also damaged, most notably above the river training wall.

- During the site visit of 13th January, a further defence was observed on the promenades rear face where there is a low level concrete wall with intermittent flood barriers, as shown in Figure 3-10.

  The flood barriers are thought to be mild steel since they are suffering extensive corrosion. The barriers function is unclear since they do not appropriately seal and many gaps exist in this defence. This suggests they are not realistically providing an improved standard of flood protection. Furthermore, the overtopping threshold is higher at the promenade front face for aware pedestrians = 0.1 l/s/m when compared to the rear face, which could be perceived as having a limiting overtopping for vehicles driving at slow speeds = 10 l/s/m (EurOtop, 2007).

  The flood barriers will help direct water away from properties and likely behave as more of a 'splash wall' to reduce overtopping on the highway (where there is a higher threshold). It is thought these barriers do prevent shingle from reaching the highway surface during events that lead to overtopping. This may expedite the ongoing shingle clearance (known to be a maintenance activity) on the promenade and improve road safety, but these flood barriers cannot be perceived as offering any improvement to the standard of protection against coastal flooding.

Figure 3-10 A low level concrete wall with intermittent flood barriers is located along the landward edge of the promenade at Sidmouth

3.11 Summary of Defence Condition

Table 3-2 summarises the assessed defence condition and residual life of each coastal defence element along each of the seven frontages as described in Sections 3.3 to 3.9.

Table 3-2 Summary of condition grade and residual life assessment for each coastal defence element

Frontage	Defence element	Condition Grade	Residual Life Estimate
1	Concrete Seawall	3 (Fair)	25-30 years (part); 45-50 years (part)
	Rock Revetment	2 (Good)	40-50 years
	Rock Groyne	3 (Fair)	-
2	Masonry Seawall	2 (Good)	50-60 years
	Walkway	1 (Very Good)	-
	Rock Revetment	2 (Good)	40-50 years
	Offshore Breakwaters	2 (Good)	-
3	Seawall	2 (Good)	75-80 years
	Rock Groyne	2 (Good)	-
4	Seawall	2 (Good)	75-80 years
	Rock Groyne	1 (Very Good)	-
5	Seawall	2 (Good)	75-80 years
	Rock Groyne	1 (Very Good)	-

Frontage	Defence element	Condition Grade	Residual Life Estimate
6	Seawall	2 (Good)	75-80 years
	River Training Wall	**4 (Poor)**	**15 years or less**
	Sheet Pile Wall	2 (Good)	40 years
	South West Water Outfall	-	-
7	Alma Bridge	1 (Very Good)	50 years
	Mass Concrete River Wall	3 (Fair)	15-30 Years

4 Analysis of Existing Defences

4.1 Overtopping

4.1.1 Introduction

One of the aims of the Sidmouth & East Beach Management Plan project, for which this defence assessment has been developed, is to "Maintain the 1990's Sidmouth Coastal Defence Scheme Standard of Service (Sidmouth Beach)." From the review of defence history relating to the 1990's scheme (see Section 2) it was apparent that there was little information presented in the available reports regarding the standard of protection against wave overtopping along the Sidmouth frontage. It is thought that the Phase I and II defences were to provide a standard of protection against coastal erosion of 1 in 50 (Royal Haskoning, 2014). Therefore, as part of this project, it was necessary to investigate wave overtopping along the Sidmouth frontage to provide a retrospective assessment of what standard was likely to have been provided by the 1990's scheme, and how that has changed in 2014 and how it may change in the future.

In order to assess the standard of protection offered by the existing Sidmouth frontage against wave overtopping (which leads to coastal flooding – further details of which are provided in the economics baseline prepared alongside this defence assessment), an overtopping assessment has therefore been carried out. This assessment has been undertaken for Frontages 1 to 6 only (refer to Figure 3-1). Frontage 7 has not been assessed due to lack of appropriate wave, tide and river flow data along the River Sid with which to undertake such analysis. However, this is something that will require further investigation in the near future as this eastern end of the Sidmouth town frontage along the River Sid, which is adjacent to the eroding cliffs to the east (East Cliff/Pennington Point) that is predicted to recede by approximately 30m or so over the next 100 years (refer to Coastal Processes Baseline report produced alongside this defences baseline), will become increasingly exposed to full coastal conditions (particularly during south-easterly storm events). Such exposure will increase the likelihood of wave overtopping leading to defence failure and thus incurrence of flood damages over time, unless the fluvial defences are upgraded to full coastal standard in advance of them becoming exposed by erosion of the cliffs. Further analysis, including numerical modelling, is required to better understand the wave and water level conditions that could be expected to occur along an exposed western wall of the River Sid defences, and in turn the associated wave overtopping potential and associated flood risk that would result.

Overtopping rates are effected by a number of considerations, such as the still water levels, wave heights, beach levels and defence crest levels. In order to provide an informative assessment, four design scenarios have been considered in the overtopping assessment. These four scenarios are summarised in Table 4-1.

Table 4-1 Summary of the four scenarios assessed in overtopping analysis

Scenario	Defence/Beach Profile Geometry assumptions	Wave and Water Level assumptions
1	Design beach profiles from the various schemes between 1990 and 1999 as detailed in Section 2 of this report for Frontages 1-6 as identified in Figure 3-1. Hard defence geometry (seawalls, revetments etc) is as constructed, beach levels are as constructed from beach recharging in these schemes.	Water level and wave heights from those presented in the *Mobile Bed Physical Model Study* (HR Wallingford, 1992, Table 4). Wave and water level combinations were established in accordance with Joint Probability Dependence Mapping and Best Practice FD2308 TR1, EA, 2003. Return periods assessed are up to 1 in 50 since wave and water levels were not provided/assessed in this report above 1 in 50.
2	As per scenario 1.	Extreme water levels and wave heights taken from (1) *Coastal Flood Boundary Conditions for UK Mainland and Islands* (Environment Agency, 2011) and (2) *Parameters for Tidal Flood Risk Assessment – Wave Parameters* (Royal Haskoning, 2012). Water levels from Environment Agency (2011) increased using current sea level rise guidance to 2014 (UKCP09, 2012). Return periods have been assessed up to 1 in 200 since current guidance allows return periods to be developed up to 1 in 200.
3	Lowest beach profiles recorded by the Plymouth Coastal Observatory (PCO) for Frontages 2-6. Frontage 1 as per scenario 1 as overtopping along this section this is not controlled by beach levels. Frontage 2 changes from a 'beach' profile to a revetment as the lowered beach exposes the revetment. Hard defence geometry (seawalls, revetments etc) is still as constructed.	As per scenario 1.
4	As per scenario 3.	As per scenario 2.

Examples of the wave and water level combinations used for Scenarios 1-4, and as described in Table 4-1, are provided in Annex H.

Assessing the combinations of beach profiles and wave and water level combinations will:

- Inform assessment of the likely design standard of protection of coastal defence works undertaken in the Phase I and Phase II schemes, as well as the Connaught Gardens and Clifton Walkway schemes (and historical schemes).
- Enable assessment of the change in standard of protection by using present day (2014) water levels (allowing for sea level rise since the 1990s) and wave heights (as opposed to the design wave and water levels used in the 1990's).
- Enable assessment of the effect of lowering beach levels on overtopping through evaluating the lowest recorded beach levels along the study frontage.

The overtopping assessment has been undertaken in accordance with best-practice guidance contained in the *Wave Overtopping of Sea Defences and Related Structures: Assessment Manual* (EurOtop, 2007). This guidance advises tolerances for overtopping depending on the function of the defence. For the Sidmouth and East BMP frontage, the following tolerances are applicable:

- Tolerable Discharge = 0.1 l/s/m for aware pedestrian who have a sea view (Table 3.2 of EurOtop, 2007)
- Damage to crest and promenade with discharge above 200 l/s/m (Table 3.5 of EurOtop, 2007).

The results of the overtopping are assessed against these criteria to identify the standard of protection offered for each frontage in terms of both public safety and structural safety.

The results from the wave overtopping analysis presented in this report are also used to undertake initial assessment of the associated coastal flood risk to Sidmouth town centre, details of which are provided in the economic baseline assessment developed alongside this report.

4.1.2 Scenario 1: Design Profile and 1992 WL & Wave data

For this Scenario 1 assessment, water levels and wave heights were used from the *Mobile Bed Physical Model Study* (HR Wallingford, 1992). However, the joint return periods were not provided within this report. In order to develop joint wave and water level combinations for return period modelling, the correlation factor method was adopted as detailed within *Joint Probability Dependence Mapping and Best Practice FD2308 TR1* (EA, 2003). Wall crest, revetment profiles, beach toe levels and beach slopes were taken from the as-built scheme drawings for the Phase I and Phase II (Figure 2-10 to 2-13) schemes and the Connaught Gardens (Figure 2-9) and Clifton Walkway (Figure 2-15) schemes, provided by EDDC. Tables 4-2, 4-3 and 4-4 show the overtopping volumes in l/s/m for each return period and frontage, and whether they pass or fail the overtopping tolerances for aware pedestrians and crest and promenade damage, outlined by EurOtop (EA, 2006) (see Section 4.1.1).

Table 4-2 Overtopping results for each return period and frontage in l/s/m (Scenario 1)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	1.037	1.402	0.001	0.000	0.000	0.000
1 in 5*	2.944	4.036	0.007	0.001	0.002	0.000*
1in 20*	7.215	9.837	0.032	0.004	0.010	26.905*
1 in 50	11.229	15.858	0.061	0.009	0.020	32.627

**Jump in overtopping due to change in overtopping caused by relative difference in beach and still water levels against the seawall (see further explanation below)*

Table 4-3 Overtopping comparison to tolerable discharges for pedestrians 0.1 l/s/m (Scenario 1)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	FAIL	FAIL	PASS	PASS	PASS	PASS
1 in 5	FAIL	FAIL	PASS	PASS	PASS	PASS
1in 20	FAIL	FAIL	PASS	PASS	PASS	FAIL
1 in 50	FAIL	FAIL	PASS	PASS	PASS	FAIL

Table 4-3 demonstrates that the as-built schemes, when assessed with the original design wave and water level conditions developed in 1992, for all return period events for Frontage 1 and 2 do not meet EurOtop guidance for overtopping limits in the vicinity of aware pedestrians. For Frontage 6, this criteria is not met for 1 in 20 and 1 in 50 return period events.

There is a noticeable jump in overtopping volumes from the 1 in 5 to 1 in 20 return period event. This is due to the overtopping function changing from wave run-up to vertical wall overtopping. When the beach toe level at the seawall is above the still water level, the beach slope is offering some reduction in wave height and the overtopping function is described as wave-run up overtopping. However, when this toe level becomes drowned by the still water level, the overtopping function changes to vertical wall overtopping, which produces much greater overtopping volumes. This highlights the importance of ensuring an adequate beach level is maintained against the seawall to reduce wave overtopping (and so coastal flooding) risk. Frontages 3 to 5 (all part of the Phase II scheme) pass the advised overtopping limits.

Table 4-4 Overtopping comparison to crest and promenade damage 200 l/s/m (Scenario 1)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	PASS	PASS	PASS	PASS	PASS
1 in 5	PASS	PASS	PASS	PASS	PASS	PASS
1in 20	PASS	PASS	PASS	PASS	PASS	PASS
1 in 50	PASS	PASS	PASS	PASS	PASS	PASS

Table 4-4 demonstrates that the as-built schemes, for all return period events and for all frontages, pass the overtopping limits for structural damage to seawall crests and promenades when assessed with the original design wave and water level conditions developed in 1992. Therefore based upon these findings it can be assumed that the original schemes constructed in the 1990's had a standard of protection in excess of 1:50 years against structural damage to the promenade and crest that could reduce the service standard of the defences.

4.1.3 Scenario 2: Design Profile and 2014 WL & Wave data

For this Scenario 2 assessment, the wave and water levels were sourced from current design guidance documentation (*Coastal Flood Boundary Conditions for UK Mainland and Islands* (Environment Agency, 2011) and *Parameters for Tidal Flood Risk Assessment – Wave Parameters* (Royal Haskoning, 2012)), and the wall crest, revetment profiles, beach toe levels and beach slopes were taken from the as-built scheme drawings for the Phase I and Phase II (Figure 2-10 to 2-13) schemes and the Connaught Gardens (Figure 2-9) and Clifton Walkway (Figure 2-15) schemes, provided by EDDC. Tables 4-5, 4-6 and 4-7 show the overtopping volumes in l/s/m for each return period and frontage, and whether they pass or fail the overtopping tolerances for aware pedestrians and crest and promenade damage, outlined by EurOtop (EA, 2006) (see Section 4.1.1).

Table 4-5 Overtopping results for each return period and frontage in l/s/m (Scenario 2)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	17.23	46.08	0.07	0.04	0.31	66.99
1 in 5	33.86	90.76	0.16	0.09	0.65	92.68
1 in 50	83.63	235.72	0.54	0.31	1.83	142.98
1 in 100	103.17	296.35	0.73	0.41	2.34	165.51
1 in 200	134.78	399.20	1.06	0.61	3.23	193.08

Table 4-6 Overtopping comparison to tolerable discharges for pedestrians 0.1 l/s/m (Scenario 2)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	FAIL	FAIL	PASS	PASS	FAIL	FAIL
1 in 5	FAIL	FAIL	FAIL	PASS	FAIL	FAIL
1 in 50	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL
1 in 100	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL
1 in 200	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL

Table 4-6 demonstrates a significant difference when compared to Table 4-3 in pass rates for all frontages and return period events, due to the different wave and water level data used for this assessment compared to Scenario 1 (refer also to Table 4-1). Nearly all frontages and return period events fail for aware pedestrian overtopping limits. The exception is Frontages 3 and 4 for low return period events. This is because the beach crest width varied along each of these frontages.

Table 4-7 Overtopping comparison to crest and promenade damage 200 l/s/m (Scenario 2)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	PASS	PASS	PASS	PASS	PASS
1 in 5	PASS	PASS	PASS	PASS	PASS	PASS
1 in 50	PASS	FAIL	PASS	PASS	PASS	PASS
1 in 100	PASS	FAIL	PASS	PASS	PASS	PASS
1 in 200	PASS	FAIL	PASS	PASS	PASS	PASS

Table 4-7 demonstrates that the majority of frontages and return period events do not fail in overtopping limits for crest and promenade damage with exception to Frontage 2, where limits are exceeded for 1 in 50, 1 in 100 and 1 in 200 return period events. This section is much lower than the adjacent frontages and its function is as a pedestrian walkway.

The masonry wall behind is the defence to coastal flooding along the Sidmouth frontage. However, this demonstrates that the walkway will have large overtopping volumes in high return period events which could lead to damage. It is noted that the construction of the walkway (shown in Section 2.9) includes a concrete slab which is further protected by block paving. Damage and uplift of the blocks could result from high return period events, though no damage was observed during the condition assessment and the block mortar was still intact.

4.1.4 Scenario 3: Lowest beach profile recorded and 1992 WL & Wave + SLR

For this Scenario 3 assessment, the same wave and water level conditions used in Scenario 1 (see Section 4.1.2) were used, but with different beach profiles against the hard-defences. Beach profile changes are only applicable to Frontages 3-6 since Frontages 1-2 are defences fronted by revetments, the revetment geometry is therefore used instead of the beach. Wall crest and revetment profiles were taken from the as-built scheme drawings for the Phase I and Phase II (Figure 2-10 to 2-13) schemes and the Connaught Gardens (Figure 2-9) and Clifton Walkway (Figure 2-15) schemes, provided by EDDC. Beach levels and slopes were taken for the lowest recorded profiles provided by the PCO monitoring between 2007 and 2014; the values used are presented below in Table 4-8. Tables 4-9, 4-10 and 4-11 show the overtopping volumes in l/s/m for each return period and frontage, and whether they pass or fail the overtopping tolerances for aware pedestrians and crest and promenade damage, outlined by EurOtop (EA, 2006) (see Section 4.1.1).

Table 4-8 Beach levels used in overtopping assessment and the associated profiles from PCO Monitoring

Frontage	Beach level at toe of wall (m AOD)	Beach level at toe of slope (m AOD)	Beach Slope	Beach Profile
3	3.2	-0.6	1 in 6	6a01451
4	2.4	-1.2	1 in 11	6a01450
5	2.4	-1.8	1 in 9	6a01445
6	1.1	-1.5	1 in 9	6a01444

Table 4-9 Overtopping results for each return period and frontage in l/s/m (Scenario 3)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	1.494	1.380	0.002	0.000	0.000	0.000
1 in 5	4.073	5.504	0.013	12.665	19.867	0.000*
1in 20	9.663	17.180	0.064	22.277	34.202	26.527*
1 in 50	14.810	29.650	0.073	24.559	37.544	29.430

**Jump in overtopping due to change in overtopping caused by relative difference in beach and still water levels against the seawall (see further explanation below)*

Table 4-10 Overtopping comparison to tolerable discharges for pedestrians 0.1 l/s/m (Scenario 3)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	FAIL	FAIL	PASS	PASS	PASS	PASS
1 in 5	FAIL	FAIL	PASS	FAIL	FAIL	PASS
1in 20	FAIL	FAIL	PASS	FAIL	FAIL	FAIL
1 in 50	FAIL	FAIL	PASS	FAIL	FAIL	FAIL

For Frontage 1, the only input to change is the still water level due to sea level rise. For Frontage 2, the overtopping scenario was adapted to model this section as a revetment. This is due to fact that during lowered beach profiles the revetment becomes exposed, whereas in the design profile the beach level extends up to the walkway crest (See Figure 2-15). Reviewing Table 4-9 compared to Table 4-2 above, it can be seen that the overtopping volumes increase when the profile is modelled for 1992 water levels and waves including SLR. This demonstrates that increased water levels do increase overtopping. A sensitivity analysis was performed to assess the effect of lowering beach levels and revetment exposure. It showed that the overtopping is more sensitive to the water level change than lowering beach levels. This is due to the revetment having an increased roughness compared to a beach. This is the same for Frontage 1 since this section is not fronted by a formal beach.

Frontage 3 still passes for the advised aware pedestrian overtopping limit, due to consistent healthy beach levels as a result of the Phase II offshore breakwaters. Frontages 4 to 6 fail in return period events above 1 in 5. This is a result of the much lower beach levels (1.2m lower) along this frontage (refer to Table 4-8 and Figure 2-9). There is a noticeable jump in overtopping volumes from the 1 in 1 to 1 in 5 return period events. When the beach toe level at the seawall is above the still water level, the beach slope is offering some reduction in wave height and the overtopping function is described as wave-run up overtopping. However, when this toe level becomes drowned by the still water level, the overtopping function changes to vertical wall overtopping, which produces much greater overtopping volumes. This again illustrates the importance of retaining adequate beach levels against the seawall (refer also to Section 4.1.2).

Table 4-11 Overtopping comparison to crest and promenade damage 200 l/s/m (Scenario 3)

	Frontage					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	PASS	PASS	PASS	PASS	PASS
1 in 5	PASS	PASS	PASS	PASS	PASS	PASS
1in 20	PASS	PASS	PASS	PASS	PASS	PASS
1 in 50	PASS	PASS	PASS	PASS	PASS	PASS

Table 4-11 demonstrates that even with the lowest recorded beach profiles, for the original scheme design wave and water level conditions developed in 1992 including SLR to 2014, all frontages pass the overtopping limits for structural damage to seawall crests and promenades for all return period events.

4.1.5 Scenario 4: Lowest beach profile recorded and 2014 WL & Wave data

For this Scenario 4 assessment, the same wave and water level conditions used in Scenario 2 (see Section 4.1.3) were used, but with different beach levels against the hard-defences. Wall crest and revetment profiles were taken from the as-built scheme drawings for the Phase I and Phase II (Figure 2-10 to 2-13) schemes and the Connaught Gardens (Figure 2-9) and Clifton Walkway (Figure 2-15) schemes, provided by EDDC. Beach levels and slopes were taken for the lowest recorded profiles provided by PCO Monitoring between 2007 and 2014, the values used are presented in Table 4-8 above. Tables 4-12, 4-13 and 4-14 show the overtopping volumes in l/s/m for each return period and frontage, and whether they pass or fail the overtopping tolerances for aware pedestrians and crest and promenade damage, outlined by EurOtop (EA, 2006) (see Section 4.1.1).

Table 4-12 Overtopping results for each return period and frontage in l/s/m (Scenario 4)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	17.23	32.78	3.43	56.73	84.28	81.79
1 in 5	33.86	74.61	5.94	77.93	114.25	108.15
1 in 50	83.63	216.33	12.97	124.05	178.59	165.83
1 in 100	103.17	275.34	15.65	139.49	199.94	185.09
1 in 200	134.78	373.42	20.00	162.97	232.32	214.34

Table 4-13 Overtopping comparison to tolerable discharges for pedestrians 0.1 l/s/m (Scenario 4)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL
1 in 5	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL
1 in 50	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL
1 in 100	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL
1 in 200	FAIL	FAIL	FAIL	FAIL	FAIL	FAIL

Table 4-13 demonstrates that all frontages fail to provide an appropriate standard of protection against overtopping for aware pedestrians when using 2014 water levels and wave heights and lowest recorded beach levels. Frontages 1 and 2 appear to not be sensitive to beach level with respect to overtopping (as discussed in Section 4.1.4). Frontages 3 to 6 demonstrate that the combination of current guidance extreme water level and wave conditions combined with low beach levels results in substantial overtopping volumes to the Sidmouth frontage.

Table 4-14 Overtopping comparison to crest and promenade damage 200 l/s/m (Scenario 4)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	PASS	PASS	PASS	PASS	PASS
1 in 5	PASS	PASS	PASS	PASS	PASS	PASS
1 in 50	PASS	FAIL	PASS	PASS	PASS	PASS
1 in 100	PASS	FAIL	PASS	PASS	PASS	PASS
1 in 200	PASS	FAIL	PASS	PASS	FAIL	FAIL

Table 4-14 shows that Frontages 1, 3 and 4 all provide a standard of protection in excess of 1 in 200 against crest and promenade damage. However, for Frontage 2 the increased water levels and waves defined by current guidance show once more a failure to provide an appropriate standard of protection against crest and promenade damage during return period events greater than 1 in 5. During an extreme 1 in 200 event, Frontages 5 and 6 show that crest and promenade damage to the promenade and crest may result if it were to be coincident with low beach levels.

4.1.6 Sensitivity of overtopping to future sea level rise

To test the sensitivity of the overtopping volumes calculated to future sea level rise, Scenarios 2 and 4 were assessed using the same input parameters, but with water levels increased by 50 and 100 years, to 2064 and 2114 respectively) based upon UKCP09 sea level rise projections (UKCP09, 2012).

Tables 4-15 to 4-18 summarise the results using the original design beach profile (refer to Table 4-1 and Section 4.1.3) in terms of overtopping volumes in l/s/m for each return period and frontage, and whether they pass or fail the overtopping tolerances crest and promenade damage, outlined by EurOtop (EA, 2006) (see Section 4.1.1) in year 50 (2064) and year 100 (2114).

From Table 4-16 it can be seen that, by 2064, Frontage 6 fails the overtopping limit for structural damage to seawall crests and promenades under the 1 in 50 event, whilst Frontage 1 fails under the 1 in 100 return period event and Frontage 2 under the 1 in 200 event. Frontages 3 to 5 all provide a standard that exceeds the 1 in 200 return period event.

Table 4-18 demonstrates that by 2114, Frontage 2 fails the overtopping limit for structural damage to seawall crests and promenades even under the 1 in 1 return period event. Frontage 6 fails this limit under the 1 in 5 event, and Frontage 1 under the 1 in 50 event. Frontages 3, 4 and 5 all pass the overtopping limit for structural damage to seawall crests and promenades for all return periods.

Table 4-15 Overtopping results for each return period and frontage in l/s/m (design profile with Scenario 2 waves and water levels; water levels increased with 50 years sea level rise to 2064)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	40.97	94.28	0.52	0.11	0.81	101.98
1 in 5	74.88	78.23	1.07	0.26	1.62	138.97
1 in 50	168.79	78.54	2.96	0.86	4.27	221.24
1 in 100	204.02	123.47	3.78	1.14	5.39	249.15
1 in 200	259.85	222.18	5.21	1.65	7.29	292.00

Table 4-16 Overtopping comparison to crest and promenade damage 200 l/s/m (design profile with Scenario 2 waves and water levels; water levels increased with 50 years sea level rise to 2064)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	PASS	PASS	PASS	PASS	PASS
1 in 5	PASS	PASS	PASS	PASS	PASS	PASS
1 in 50	PASS	PASS	PASS	PASS	PASS	FAIL
1 in 100	FAIL	PASS	PASS	PASS	PASS	FAIL
1 in 200	FAIL	FAIL	PASS	PASS	PASS	FAIL

Table 4-17 Overtopping results for each return period and frontage in l/s/m (design profile with Scenario 2 waves and water levels; water levels increased with 100 years sea level rise to 2114)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	106.37	289.49	1.67	0.44	2.46	168.69
1 in 5	180.45	241.08	3.26	0.96	4.66	230.70
1 in 50	369.79	552.18	8.48	2.88	11.47	374.06
1 in 100	437.45	837.16	10.70	3.75	14.25	424.04
1 in 200	542.46	1439.59	14.47	5.30	18.89	502.03

Table 4-18 Overtopping comparison to crest and promenade damage 200 l/s/m (design profile with Scenario 2 waves and water levels; water levels increased with 100 years sea level rise to 2114)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	FAIL	PASS	PASS	PASS	PASS
1 in 5	PASS	FAIL	PASS	PASS	PASS	FAIL
1 in 50	FAIL	FAIL	PASS	PASS	PASS	FAIL
1 in 100	FAIL	FAIL	PASS	PASS	PASS	FAIL
1 in 200	FAIL	FAIL	PASS	PASS	PASS	FAIL

Tables 4-19 to 4-22 summarise the results using the lowest beach profile for each frontage based on PCO monitoring data between 2007 and 2014 (refer to Table 4-8) in terms of overtopping volumes in l/s/m for each return period and frontage, and whether they pass or fail the overtopping tolerances crest and promenade damage, outlined by EurOtop (EA, 2006) (see Section 4.1.1) in year 50 and year 100.

Table 4-20 shows that by 2064, Frontages 2, 5 and 6 fail the overtopping limit for structural damage to seawall crests and promenades under the 1 in 50 event, whilst Frontages 1 and 4 fail under the 1 in 100 return period event. Only Frontage 3 provides a standard that exceeds the 1 in 200 return period event for aware pedestrians.

Table 4-22 demonstrates that by 2114, Frontages 2 and 5 fail the overtopping limit for structural damage to seawall crests and promenades even under the 1 in 1 return period event. Frontage 6 fails this limit under the 1 in 5 event, and Frontages 1 and 4 under the 1 in 50 event. Only Frontage 3 passes the overtopping limit for structural damage to seawall crests and promenades for all return periods.

Table 4-19 Overtopping results for each return period and frontage in l/s/m (lowest beach profile with Scenario 2 waves and water levels; water levels increased with 50 years sea level rise to 2064)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	40.97	94.28	7.03	85.64	125.21	117.72
1 in 5	74.88	191.56	11.83	116.86	168.78	156.80
1 in 50	168.79	484.51	24.80	186.91	265.44	244.09
1 in 100	204.02	598.72	29.66	210.79	298.15	273.72
1 in 200	259.85	783.15	37.43	247.49	348.29	319.15

Table 4-20 Overtopping comparison to crest and promenade damage 200 l/s/m (lowest beach Scenario 2 waves and water levels; water levels increased with 50 years sea level rise to 2064)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	PASS	PASS	PASS	PASS	PASS
1 in 5	PASS	PASS	PASS	PASS	PASS	PASS
1 in 50	PASS	FAIL	PASS	PASS	FAIL	FAIL
1 in 100	FAIL	FAIL	PASS	FAIL	FAIL	FAIL
1 in 200	FAIL	FAIL	PASS	FAIL	FAIL	FAIL

Table 4-21 Overtopping results for each return period and frontage in l/s/m (lowest beach with Scenario 2 waves and water levels; water levels increased with 100 years sea level rise to 2114)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	106.37	289.49	16.33	141.98	203.84	188.16
1 in 5	180.45	526.50	26.58	194.86	276.66	253.98
1 in 50	369.79	1162.77	53.28	317.77	444.31	405.92
1 in 100	437.45	1394.84	63.05	360.77	502.59	458.82
1 in 200	542.46	1759.14	78.54	427.96	593.40	541.28

Table 4-22 Overtopping comparison to crest and promenade damage 200 l/s/m (lowest beach with Scenario 2 waves and water levels; water levels increased with 100 years sea level rise to 2114)

	Profile Number					
Return Period	**1**	**2**	**3**	**4**	**5**	**6**
1 in 1	PASS	FAIL	PASS	PASS	FAIL	PASS
1 in 5	PASS	FAIL	PASS	PASS	FAIL	FAIL
1 in 50	FAIL	FAIL	PASS	FAIL	FAIL	FAIL
1 in 100	FAIL	FAIL	PASS	FAIL	FAIL	FAIL
1 in 200	FAIL	FAIL	PASS	FAIL	FAIL	FAIL

With reference to Table 4-18 in comparison to Tables 4-4 and 4-7, this sensitivity testing illustrates that as sea levels rise, if a beach that matches the 1990's design profile can be retained along the frontage, the central part of Sidmouth's coastal defences across Frontages 3, 4 and 5 should continue to provide the standard of protection of the order of the original scheme designed and constructed in the 1990s. However, Frontages 1, 2 and 6 will not be able to provide the same standard in the future.

Comparing Table 4-22 to Tables 4-4 and 4-7, it is clear that if beach levels are allowed to lower along the frontage, then the coastal defences at Sidmouth will become increasingly vulnerable to wave overtopping leading to structural failure of the defences.

4.1.7 Summary of overtopping and further observations

The following key conclusions can be drawn from the overtopping assessment presented in Sections 4.1.2 to 4.1.6:

1. The Connaught Garden (and historical scheme) and Clifton Walkway Schemes provide a standard of protection of less than a 1 in 1 return period for pedestrian safety, for both 1992 and 2014 water level and wave data
2. Overtopping for Frontages 1 and 2 is not adversely affected by beach levels. Beach levels are negligible along Chit Rocks, whereas along Clifton Walkway lowering beach levels expose a revetment which has a positive effect on reducing overtopping volumes. However, it should be noted that the reduction in wave energy created by the offshore breakwaters cannot be modelled with this approach. Physical or numerical modelling would be required to assess the effects on wave energy and the associated reduction in overtopping.
3. Overtopping at Frontages 3, 4, 5 and 6 is controlled by the beach levels in front of the seawall. Importantly, when the still water level exceeds the beach level at the toe of the wall, the overtopping function changes to vertical wall overtopping which leads to much greater overtopping volumes along these frontages. This highlights the importance of ensuring an adequate beach level is maintained against the seawall to reduce wave overtopping (and so coastal flooding) risk.
4. The Phase II scheme provided a standard of protection against overtopping that causes a risk of damage to the crest and promenade of greater than 1 in 50, based on the original design water level and wave data provided by HR Wallingford in 1992 and the design beach profile. This excludes Frontage 6 where overtopping limits were exceeded in 1 in 20 and 1 in 50 events, although this frontage was not subject to beach recharge in the Phase II scheme. Estimated beach levels during 1992 suggest this frontage has much lower levels than those post recharging on adjacent Frontages (approximately 1m lower at the seawall), and this is the cause of overtopping limits being exceeded at Frontage 6.
5. At present day (2014); with current guidance extreme wave and water levels and the 1990s design beach profile, the Phase II scheme is assessed as actually providing a standard of protection against structural damage to the seawall crest and promenade in excess of 1 in 200.
6. The standard of protection against crest and promenade damage is reduced at present due to the low beach levels along much of the beach management plan frontage. Frontage 2 is particularly susceptible in 2014 to crest and promenade damage due to its low crest level leading to high overtopping volumes. Frontages 5 to 6 could be susceptible to crest and promenade damage in 1 in 200 extreme events, based on current guidance extreme wave and water levels and lowered beach profiles. It is noted that this assessment could not quantify the reduction in overtopping volumes due to the presence of the offshore breakwaters in front of the Frontage. Though it can be seen in Figure 4-1 it is likely of little benefit.
7. As sea levels rise, the standard of protection will continue to reduce for the entire study frontage. For frontages 3-5, the impact could be reduced if beach levels approaching the 1990's design beach profile can be retained.
8. Frontage 7 has not been assessed due to lack of appropriate wave and water level data along this part of the frontage. Detailed modelling of wave and water levels around this area would be required to assess the potential impact of increased wave loading (and so increased possibility of flood risk to Sidmouth town centre) of the River Sid wall up to Ham Weir as it becomes more exposed as the adjacent East Cliff/Pennington Point frontage erodes further in the future. The combination of the tidal level, river level and wave climate make this a more complex assessment. A topographic survey of the full defence profile would be required to assess the risk of pedestrian overtopping (at the water edge of the wall) and flooding (at the landward edge of the wall) since they have changing crest levels.

Further to the analysis above, images were obtained from the recent storms on 3rd February 2014. These events were estimated to be extreme, in the order of 1 in 100 to 1 in 200 return period events. Figure 4-1 demonstrates the results of the overtopping analysis well. It can be seen that Frontage 2 is extensively overtopping during this event, whilst Frontage 3 is not overtopping due to the high beach levels. In the distance of the photo it can be seen that Frontages 4 to 6 are leading to overtopping with impulsive conditions, this is due to the lower beach levels when compared to Frontage 3. This further validates the results of the overtopping analysis. Figure 4-2 shows Frontage 1 extensively overtopping during the storm, further validating the overtopping analysis.

Figure 4-1 Overtopping during February storms of Frontages 2-6

Figure 4-2 Overtopping during February storms of Frontage 1

4.2 Scour and Undermining

In general the Sidmouth frontage has a very low risk of defence failures due to scour and undermining. This is due to the extensive construction works over the years which have provided scour and undermining protection measures throughout the frontage (refer to Section 2). Each defence length is individually discussed below.

4.2.1 Frontage 1: Jacobs Ladder to Clifton Walkway

This frontage includes a seawall ramp in a poor state in need of immediate repairs and a seawall fronted by a rock revetment. The rock revetment was installed to address undermining issues (1994) and was constructed on top of bedrock with the bottom rock keyed into the bedrock layer (See Figure 2-8).

It is not possible to comment definitively on the condition of the existing toe of the seawall due to the presence of the revetment. However, it is assumed the structure is stable as there are no signs of structural movement in the seawall. For these reasons the scour and undermining risk is considered to be very low. If displaced rocks or sliding of the revetment does occur, appropriate maintenance works should be undertaken to preserve the revetment which forms the first line of defence to scour and undermining.

4.2.2 Frontage 2: Clifton Walkway to West Pier

In this section of frontage, a masonry seawall is fronted by a concrete walkway which was constructed on top of a revetment built on bedrock topped by a revetment with smaller rock grading. The beach levels are healthy along this frontage due to the presence of the nearby offshore breakwaters. Once more this frontage is heavily protected from scour and undermining. For these reasons the scour and undermining risk is considered to be very low. Unfortunately there have been no as-built or post construction surveys of the offshore rock groynes and it is therefore not possible to comment on the scour and undermining risk posed to the structures. It is recommended a baseline profile survey is undertaken in the short term and regular surveys every 5 years to continue assessing the structures stability.

4.2.3 Frontage 3: West Pier to Bedford Steps Groyne

This frontage includes a masonry seawall which was underpinned by mass concrete and fronted by a mass concrete apron with down toe constructed into bedrock. A rock revetment has been placed in front of the apron and keyed into the bedrock during Phase I and the 1993 Emergency Works.

The condition of the apron was not able to be evaluated due to high beach levels burying the apron. However, the risk of undermining and scour remains very low as the previous measures have mitigated this risk. The exposure of the revetment below the recharged beach may be of concern from an amenity perspective, occurring at a level of approximately 1.15mOD for chainage 340m-480m and 2.00mOD from chainage 480-540m (refer to Figures 2-3 and 2-6).

4.2.4 Frontage 4: Bedford Steps Groyne to York Steps Groyne

This frontage, like Frontage 3, includes a masonry seawall which was underpinned by mass concrete and fronted by a mass concrete apron with down toe constructed into bedrock. A rock revetment has been placed in front of the apron and keyed into the bedrock during Phase I and the 1993 Emergency Works.

The condition of the apron was assessed and shown to have cracking, damaged coping and suffering severe abrasion. A repair is recommended to protect the asset which forms part of the defence to undermining and scour.

The risk of undermining and scour remains very low as the defence measures have mitigated the risk. The exposure of the revetment below the recharged beach may be of concern from an amenity perspective, occurring at a level of approximately 2.00mOD. Bedford Steps groyne was also laid onto bedrock.

4.2.5 Frontage 5: York Steps Groyne to East Pier Groyne

This frontage, like Frontages 3 and 4, includes a masonry seawall which was underpinned by mass concrete and fronted by a mass concrete apron with down toe constructed into bedrock. The condition of the apron was assessed and shown to have cracking, spalling, damaged coping and suffering severe abrasion. This apron is in poor condition and is recommended to be repaired in the near future. The repair will protect the asset which forms the only defence to undermining and scour in this frontage.

The risk of undermining and scour remains very low as the apron still has a down toe into bedrock and the beach is also currently providing protection. The condition of the apron at the toe was not able to be assessed. York Steps groyne was also laid onto bedrock.

4.2.6 Frontage 6: East Pier Groyne to Alma Bridge

This frontage includes a reinforced concrete faced seawall with reinforced concrete down toe. There is no revetment or lower level seawall (apron). The protection from undermining and scour to the seawall is provided by the reinforced concrete erosion toe which was extended into bedrock, as well as the beach. The risk of undermining remains low. However, reduced beach levels leading to erosive forces on the bedrock could lead to an undermining risk if the exposure is long term. An approximate level of -0.60mOD (approximate bedrock level) at the foot of the wall is suggested as a trigger level for remedial works to prevent long term exposure and scour of the bedrock at the seawall footing.

The river training wall is in poor condition and needs to be repaired in the near future (refer also to Section3.8.2). The wall is exhibiting scour and undermining on the River Sid side, large cracking, holes and structural movement. The risk of scour to both sides of the training wall needs to be evaluated. At present, data on the training wall construction to undertake more detailed assessment of this risk is not available. However, there is signs of significant scour on the river side of the training wall and it is strongly recommended that scour protection be provided as a minimum in the short term.

The river training wall also has a large potential for retaining material and, under full height beach retention, it may lead to failure. The wall is also subject to high wave loading as witnessed in the February 2014 storm events. A full structural assessment of this wall is recommended to assess what beach level is acceptable before the wall becomes unstable and what wave forces can be tolerated. This would inform necessary repair works to the structure.

4.2.7 Frontage 7: River Sid: Alma Bridge to Ham Weir

No data is currently available to undertake assessment of undermining risk along this section of frontage. Undermining and scour was observed on the day of the site visit on 13th January 2014. No movement was observed in the existing walls. There are a number of difficulties in estimating the risk posed to this wall by undermining and scour. The main risks are:

- the existing scour holes increasing in width/depth such that the wall spanning the hole suffers a slump failure; or
- the increase in pressure due to the reduced contact area causes a bearing pressure failure of the rock mass.

In order to assess these risks the following information would be usefully acquired:

1. Construction details of the existing wall – without these details it is very difficult to appreciate how scour and undermining will impact on the structural stability of the wall (the walls structural response will be related to its construction). If visual surveys reveal slump and heave of the promenade, this could be due to scour.
2. Flow rates, velocities and water depths for a range of flow conditions (both high and low) – Rock scour calculations can be undertaken to estimate the erosion rate of the rock layer. Primary inputs are the flow rates and velocities and water pressure, the critical condition could be at low water or high water depths.
3. Survey of existing scour profile – this can be used to establish the existing scour risk. Survey to include scour profile along the length of the wall and measurements of scour depths. This can then be used to approximate the future risk of scour and undermining.

Due to the uncertainties with estimating the scour and undermining risk of the rock mass, it may not be cost efficient to assess the risk of scour and instead, installation of scour protection measures could be a preferable approach to managing the risk.

5 Conclusions

Having undertaken the condition assessment of this frontage and examined the extensive documentation for the various schemes of construction, there are a number of important conclusions to be made. This has been subdivided for ease of reference in the following sections.

5.1 Defence Condition

Generally the condition of the existing defence assets is good, particularly those constructed during the Phase I and II schemes during the 1990s. However, there are a few key defences which it is recommended be considered for remedial works during the short term. These are as follows:

1. The access ramp at Jacobs Ladder Beach has exposed reinforcement and if a repair is not undertaken in the shorter term then the structure will deteriorate beyond repair, estimated to be within the next 15 years.
2. Expansion joints to walls and promenade slabs should be repaired and/or replaced by an appropriate Contractor and a suitable product. At present, many joints have degraded, some have been filled with mortar. This will lead to degradation of joints and deterioration of the wall in long term through spalling.
3. There is no available information for accurately assessing the condition of the offshore breakwaters. Whilst they do not appear to be structurally failing it is strongly recommended that a baseline survey is undertaken of the breakwaters and regular surveys at five year intervals to establish any structural movement and highlight any scour or undermining to the structures.
4. The river training wall is suffering severe scour at the toe on the river side. The wall is also suffering deterioration from scour and abrasion and holes and cracks have developed. It is recommended that immediate action is taken to stabilise this training wall, scour protection to the toe is recommended and concrete patch repairs to the cracks and holes which have developed.
5. The River Sid wall has deep penetrating scour holes developing at the wall formation level and along construction joints. Scour protection to the toe of the wall worst affected is recommended and consideration to concrete repairs to the face of the wall. The severity of the defects are difficult to quantify since details of the wall construction are not available. It is further recommended that some investigation is undertaken to obtain any information or records on the wall construction.
6. It is unclear as to the purpose of the mild steel flood barriers but consideration to their function and potential replacement/improvements/removal is recommended. At present, the existing barriers are corroding and not sealed, it is also uncertain whether they all open and close.

Note, assessment of performance of beach recharge is discussed in the Coastal Processes Baseline report developed alongside this report.

5.2 Overtopping

It is important to discuss the relevance of the overtopping analysis further to the points summarised in Section 4.1.6. There appears to be poor documentation on the standard of protection offered by the various defences which have been built along the Sidmouth frontage. It is thought that the Phase I and II defences were to provide a standard of protection against coastal erosion of 1 in 50 (Royal Haskoning, 2014). The analysis presented in Section 4 indicates that using the 1990's scheme design data (Scenario 1) this would appear to be the case, whilst analysis for Scenario 2 suggests the standard of protection against wave overtopping that results in damage to seawall crest and promenade is greater than 1 in 200.

The overtopping analysis has also demonstrated that the effects of sea level rise and current guidance on determining extreme water levels and waves has a major reduction in standard of protection for aware pedestrians compared to use of the original 1990s scheme design waves and water levels. When assessing the Phase I and II design profiles with the water levels derived in the same time frame, the standard of protection was assessed to be >1 in 50 in 2014, based on the threshold being for aware pedestrians. When comparing this to the same design profiles with Scenario 2 (current guidance) wave and water levels, the standard of protection for aware pedestrians drops to 1 in 5 maximum (refer to Table 4-6). This demonstrates that the standard of protection for aware pedestrians is significantly declining due to the effects of sea level rise over the next 50 to 100 years (to 2114). The threshold could be considered to be higher if the promenade could be formally closed off from the public, such that the tolerable discharge is reflecting 10 l/s/m for slow driving as opposed to 0.1 l/s/m for aware pedestrians. In this instance, for the majority of frontages (with exception to the eastern end of the seawall), the schemes provide an adequate standard of protection with 2014 water levels.

Frontage 1 and 2 were found not to be sensitive to beach levels. However, Frontages 3 to 6 have shown to be considerably effected by the beach levels along the frontage. This is emphasised by the change in overtopping function. When the beach toe level at the seawall is above the still water level, the beach slope is offering some reduction in wave height and the overtopping function is described as wave-run up overtopping. However, when this toe level becomes drowned by the still water level, the overtopping function changes to vertical wall overtopping, which produces much greater overtopping volumes. This demonstrates that both increasing sea levels and decreasing beach levels lead to serious increases in overtopping volumes along Sidmouth frontage. Consideration to trigger levels for recycling, flood warning procedures and defence options is highly recommended and a more detailed investigation undertaken to better quantify and address these risks.

As sea levels rise, the amount of overtopping along the frontage will increase and so the standard of protection against wave overtopping that causes damage to seawall crest and promenade will reduce to typically <1 in 50 by 2064 and <1in 5 by 2114, unless beach levels approaching the 1990's design beach profile can be retained along much of the frontage. Therefore future management options that either retain more beach along the frontage, or reduce the impact of higher overtopping, will need to be considered.

Lastly, the assessment has demonstrated that of all the frontages which were constructed during the Phase I, II and III schemes in the 1990s, Frontage 6 poses the greatest risk for overtopping. This frontage shows large overtopping volumes in all scenarios modelled. This is because the beach levels are much lower than adjacent frontages. This is not just shown in the overtopping analysis but also in storm video footage from February 2014 storms, where overtopping volumes were so large that the asphalt on the promenade was severely damaged. This was also due to the presence of the river training wall which caused reflection of the waves on to the promenade and wave run-up over the concrete slipway in this area, neither of which can be represented in the overtopping techniques available and applied in this work. Physical modelling of this area would be the only way to investigate the wave overtopping risk in this area in a robust way.

5.3 Scour and Undermining

The coastal defences at Sidmouth are at very low risk of failure and damage resulting from scour and undermining as a result of the extensive scour protection constructed over the various phases.

The river wall and river training wall are both suffering the effects of scour and undermining and should be considered for remedial works in the short term to address this risk. The river wall is deemed as particularly unstable as the structure is showing signs of structural movement and is behaving as a wave wall and retaining structure.

For the river wall, the main risks are the existing scour holes increasing in width/depth such that the wall spanning the hole suffers a slump failure, or the increase in pressure due to the reduced contact area causes a bearing pressure failure of the rock mass. In order to assess these risks the following information would be usefully acquired:

1. Construction details of the existing wall.
2. Flow rates, velocities and water depths for a range of flow conditions (both high and low).

3. Survey of existing scour profile to include scour profile along the length of the wall and measurements of scour depths.

Due to the uncertainties with estimating the scour and undermining risk of the rock mass, it may not be cost efficient to assess the risk of scour and instead, installation of scour protection measures could be a preferable approach to managing the risk.

With regards the offshore rock breakwaters, unfortunately there have been no as-built or post construction surveys of these structrues and it is therefore not possible to comment on the scour and undermining risk posed to the structures. It is recommended a baseline profile survey is undertaken in the short term and regular surveys every 5 years to continue assessing the structures stability.

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Annexes A to G: Site Inspection Photos (13/01/2014)

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Annex A1: Frontage 1 - Seawall

Figure A1: Corrosion holes in hand railing

Figure A2: Exposed reinforcement and abrasion

Figure A3: Blocked outfall

Figure A4: Damaged seawall coping

Figure A5: Buried life ring

Figure A6: Scoured promenade

Figure A7: Corroded hand railing

Figure A8: Cracking at drainage outlet

Figure A9: Coping damage

Figure A10: Spalling at expansion joints

Figure A11: Promenade joints without sealant

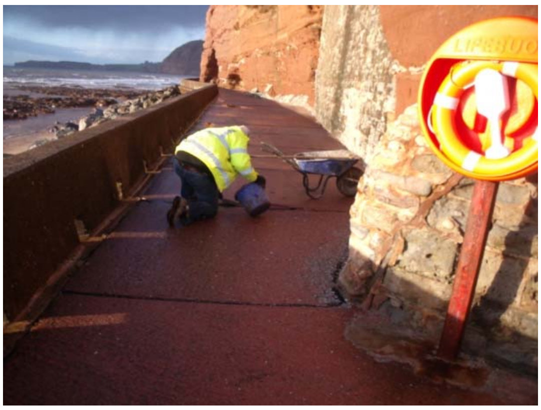

Figure A12: Mortar filling to expansion joints

Annex A2: Frontage 1 – Revetment

Figure A13: Revetment in front of seawall

Figure A14: Large voids between rock armour

Annex A3: Frontage 1 – Rock Groyne

Figure A15: West groyne flank, displaced rocks

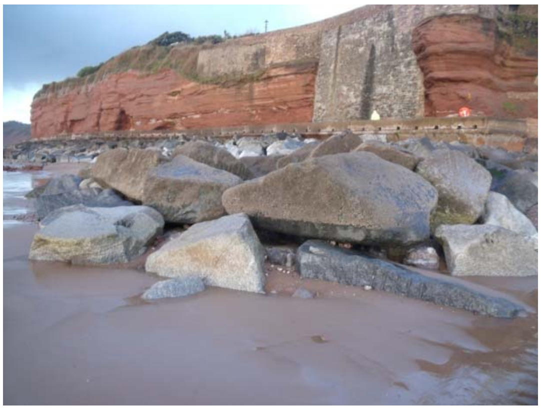

Figure A16: Rock groyne packing and profile

Figure A17: East groyne flank, displaced rocks

Figure A18: Large displaced rocks

Annex B1: Frontage 2 – Seawall

 ***Figure B1: Masonry seawall***	 ***Figure B2: Masonry seawall***
 ***Figure B3: Masonry mortar repairs***	 ***Figure B4: Walkway access ramp***

Annex B2: Frontage 2 – Walkway

Figure B5: Outfall and screen

Figure B6: Debris and shingle on walkway

Figure B7: Blocked gully

Figure B8: West Pier access ramp

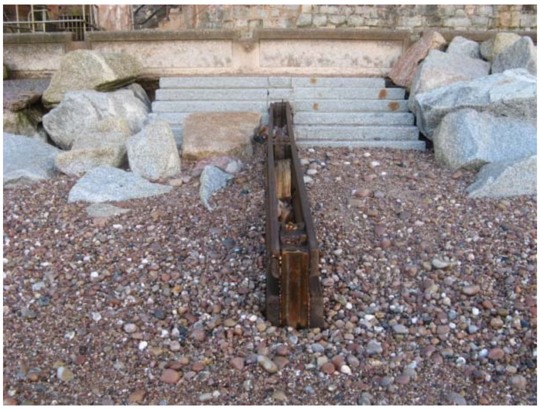

Figure B9: Remaining groyne

Figure B10: Walkway edging

Annex B3: Frontage 2 – Revetment

Figure B11: Rock revetment

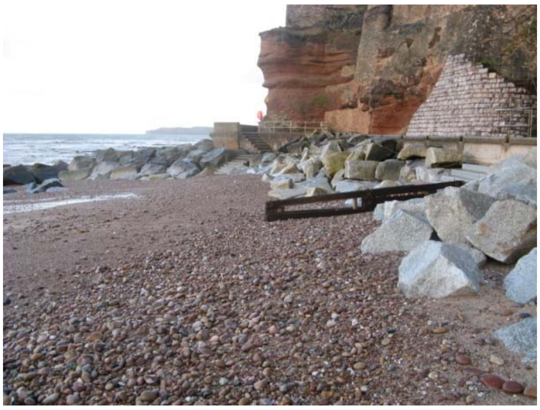

Figure B12: Rock revetment

Figure B13: Displaced rocks and shingle infill at toe

Figure B14: Displaced rocks

Annex B4: Frontage 2 – Offshore Breakwaters

Figure B15: Eastern breakwater

Figure B16: Western breakwater

Figure B17: Displaced rocks on western breakwater

Figure B18: Rock packing on western breakwater

Annex C1: Frontage 3 – Seawall

Figure C1: Masonry wall with RC coping

Figure C2: Hand railing free of corrosion

Figure C3: Large outfall and water ponding

Figure C4: Access stairs and moored boats

Figure C5: Cracking and missing sealant on coping

Figure C6: Blocked outfalls

Annex C2: Frontage 3 – Rock Groyne

Figure C7: Bedford Steps rock groyne

Figure C8: Single infilling and small rocks

Figure C9: Consistent crest

Figure C10: Small rocks used to plug voids

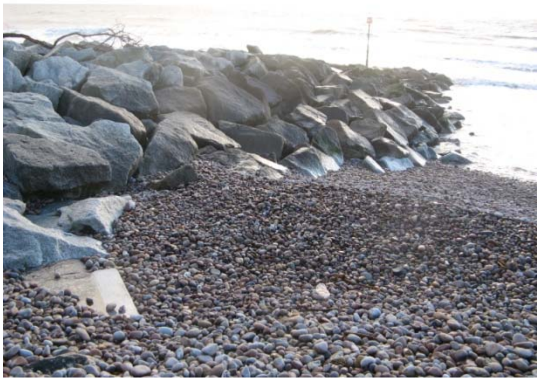

Figure C11: Rock groyne West flank

Figure C12: Rock groyne East flank

Annex D1: Frontage 4 – Seawall

Figure D1: Mortar repairs to apron

Figure D2: Blocked outfall

Figure D3: Spalling at expansion joints

Figure D4: Masonry joints filled with mortar

Figure D5: Cracking and abrasion to apron

Figure D6: Access ramp

Annex D2: Frontage 3 – Rock Groyne

Figure D7: Rock groyne

Figure D8: Rock groyne West flank

Figure D9: Rock groyne East flank

Figure D10: Rock packing and angularity

Annex E1: Frontage 5 – Seawall

Figure E1: Badly damaged apron

Figure E2: Badly damaged apron

Figure E3: Exposed protruding reinforcement

Figure E4: Seawall to West

Figure E5: Blocked outfall

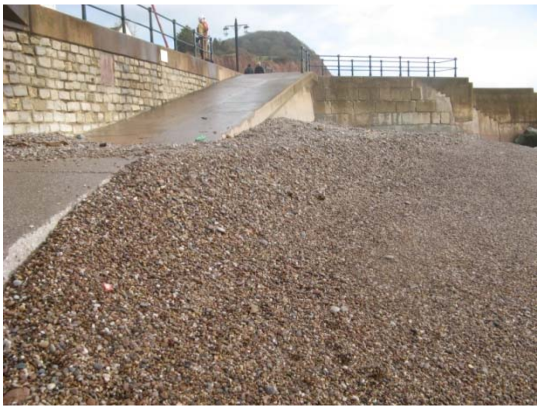

Figure E6: East Pier ramp

Annex E2: Frontage 5 – Rock Groyne

Figure E7: Rock groyne West flank

Figure E8: Rock groyne crest

Figure E9: Rock groyne East flank

Figure E10: Concrete slipway

Figure E11: East Pier

Figure E12: East Pier concrete block joints

Annex F1: Frontage 6 – Seawall

<table>
<tr>
<td>
Figure F1: RC faced seawall</td>
<td>
Figure F2: RC wall construction joint</td>
</tr>
<tr>
<td>
Figure F3: West access ramp</td>
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Figure F4: East access ramp</td>
</tr>
</table>

Annex F2: Frontage 6 – River Training Wall

Figure F5: River training wall

Figure F6: Exposed manhole ring

Figure F7: Holes and cracking

Figure F8: Extensive, later construction

Figure F9: West groyne flank, no hand railing

Figure F10: East groyne flank

Annex F3: Frontage 6 – Sheet Pile Wall

Figure F11: Sheet pile wall

Figure F12: Sheet pile close up

Figure F13: Sheet pile wall mass concrete scour

Figure F13: Sheet pile wall mass concrete scour

Annex G1: Frontage 7 – Alma Bridge

Figure G1: Alma Bridge

Figure G2: Bridge Footings

Annex G2: Frontage 7 – Mass Concrete River Wall

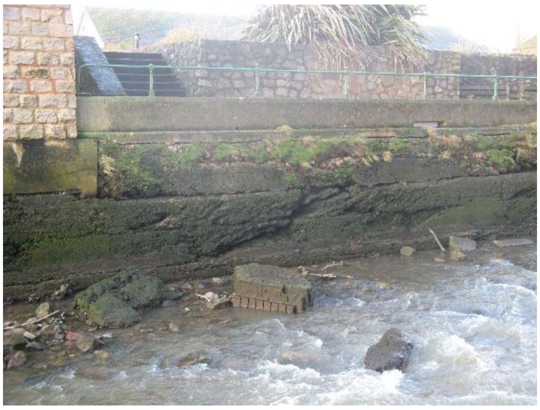

Figure G5: River wall

Figure G6: River wall spalling/erosion

Figure G7: River wall spalling/erosion

Figure G8: River wall undermining

Figure G9: River wall undermining

Figure G10: River wall walkway

Annex H: Wave and Water Level Data for Overtopping Analysis

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Annex H: Wave & Water Level Data for Overtopping Analysis

075° - 105°				
JP Return Period	**Water Level (mOD)**	**Return Period**	**Wave Height, Hs (m)**	**Period, Tz (s)**
1	2.22	1	2.34	5.50
5	2.39	1	2.34	5.50
	2.25	5	3.10	6.30
20	2.56	1	2.34	5.50
	2.40	5	3.10	6.30
	2.28	20	3.76	6.90
50	2.65	1	2.34	5.50
	2.35	5	3.10	6.30
	2.26	20	3.76	6.90
	2.09	50	4.19	7.30
105° - 135°				
JP Return Period	**Water Level (mOD)**	**Return Period**	**Wave Height, Hs (m)**	**Period, Tz (s)**
1	2.22	1	3.18	6.40
5	2.39	1	3.18	6.40
	2.25	5	4.19	7.30
20	2.56	1	3.18	6.40
	2.40	5	4.19	7.30
	2.28	20	5.02	8.00
50	2.65	1	3.18	6.40
	2.35	5	4.19	7.30
	2.26	20	5.02	8.00
	2.09	50	5.57	8.40
135° - 165°				
JP Return Period	**Water Level (mOD)**	**Return Period**	**Wave Height, Hs (m)**	**Period, Tz (s)**
1	2.22	1	2.77	6.00
5	2.39	1	2.77	6.00
	2.25	5	3.70	6.90
20	2.56	1	2.77	6.00
	2.40	5	3.70	6.90
	2.28	20	4.44	7.50
50	2.65	1	2.77	6.00
	2.35	5	3.70	6.90
	2.26	20	4.44	7.50
	2.09	50	4.91	7.90
165° - 195°				
JP Return Period	**Water Level (mOD)**	**Return Period**	**Wave Height, Hs (m)**	**Period, Tz (s)**
1	2.22	1	3.37	6.60
5	2.39	1	3.37	6.60
	2.25	5	4.14	7.30
20	2.56	1	3.37	6.60
	2.40	5	4.14	7.30
	2.28	20	4.74	7.80
50	2.65	1	3.37	6.60
	2.35	5	4.14	7.30
	2.26	20	4.74	7.80

Annex H: Wave & Water Level Data for Overtopping Analysis

JP Return Period	Water Level (mOD)	Return Period	Wave Height, Hs (m)	Period, Tz (s)
	2.09	50	5.11	8.10
195° - 225°				
JP Return Period	**Water Level (mOD)**	**Return Period**	**Wave Height, Hs (m)**	**Period, Tz (s)**
1	2.22	1	2.46	5.60
5	2.39	1	2.46	5.60
	2.25	5	3.00	6.20
20	2.56	1	2.46	5.60
	2.40	5	3.00	6.20
	2.28	20	3.44	6.60
50	2.65	1	2.46	5.60
	2.35	5	3.00	6.20
	2.26	20	3.44	6.60
	2.09	50	3.72	6.90
225° - 255°				
JP Return Period	**Water Level (mOD)**	**Return Period**	**Wave Height, Hs (m)**	**Period, Tz (s)**
1	2.22	1	1.07	3.70
5	2.39	1	1.07	3.70
	2.25	5	1.37	4.20
20	2.56	1	1.07	3.70
	2.40	5	1.37	4.20
	2.28	20	1.62	4.60
50	2.65	1	1.07	3.70
	2.35	5	1.37	4.20
	2.26	20	1.62	4.60
	2.09	50	1.78	4.80

Table H-1 – Joint return periods used for Scenario 1. For scenario 3 water levels were increased in accordance with current SLR guidance (see main report, Section 4).

	Joint exceedence return period (years)							
	1	2	5	10	20	50	100	200
EWLs (mOD)	Offshore resultant wave height off Sidmouth (m) from South East direction							
2.49	3.77	3.93	4.15	4.30	4.43	4.52	4.58	4.62
2.52	3.68	3.84	4.06	4.22	4.36	4.49	4.55	4.60
2.64	3.47	3.63	3.84	4.01	4.17	4.36	4.47	4.53
2.72	3.34	3.50	3.71	3.88	4.04	4.26	4.39	4.48
2.78	#N/A	3.37	3.58	3.75	3.91	4.13	4.29	4.41
2.88	#N/A	#N/A	3.41	3.58	3.74	3.95	4.12	4.28
2.95	#N/A	#N/A	#N/A	3.45	3.61	3.82	3.99	4.15
3.04	#N/A	#N/A	#N/A	#N/A	#N/A	3.65	3.81	3.98
3.12	#N/A	#N/A	#N/A	#N/A	#N/A	3.52	3.68	3.85
3.15	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	3.61	3.77
3.18	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	3.55	3.72
3.26	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	3.59
3.37	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
3.44	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Annex H: Wave & Water Level Data for Overtopping Analysis

Figure H-2 – Example of joint return periods used for Scenario 2 and 4 from 135 degrees (see main report, Section 4).