Technical Note

HaskoningDHV UK Ltd.
Water & Maritime

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From:	Guillermo Ingelmo de Tomas
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Our reference:	PC1679-RHD-ZZ-XX-FN-Z-0003
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Subject: **Sidmouth BMS – Breakwaters Concept Design**

1 Introduction

1.1 Background

Following the updated Partnership Funding Calculator for FCRM projects by the Environment Agency in 2020, additional FDGiA was released for the approved flood defence scheme as shown in the draft OBC for Sidmouth, which provided the basis for exploring alternative options. Therefore, East Devon District Council (EDDC), in collaboration with an elected Stakeholders Sub-Group, have requested Royal HaskoningDHV (RHDHV) to undertake a high-level assessment of additional flood defence options, including but not limited to, options that were previously discounted during the development of the Beach Management Plan (BMP) for the main town (Frontage B) and East Beach (Frontage C), see Figure 1-1.

Error! Reference source not found. shows an aerial image of the scheme in which the different areas of the projects have been identified.

Figure 1-1 : Aerial image of Sidmouth's frontage

1.2 Project Objectives

The objectives of the Sidmouth Flood Defence Scheme are as follows (as per “Pause Study Scope V8” by EDDC):

1. Maintain the 1990’s Sidmouth Coastal Defence Standard of Service (Sidmouth Beach);
2. Reduce the rate of beach and cliff erosion to the east of the R Sid (East Beach).
3. Carry out (1) and (2) in an integrated, justifiable and sustainable way

The scheme described in this document have been conceived to meet all the above objectives. In addition, construction costs, including elements outside the scope of this exercise and risks need to be between £10M and £12M.

It is worth noting that at this stage, no Landscape Visual Impact Assessment (LVIA) or environmental impact assessments were instructed for this exercise.

1.3 Purpose of the Document

The purpose of this technical note is to summarise the design process behind the assessment of the alternative options for Sidmouth Flood Defences Scheme, providing information on the technical and non-technical considerations behind the proposed schemes.

2 Proposed Schemes

This section provides information about the proposed schemes and the conclusions reached from technical evaluation during this stage.

2.1 Long groyne to the west of River Sid

The possibility of implementing a long groyne immediately to the west of the river Sid’s training wall was evaluated and considered to have only limited success in meeting the objectives of the project.

From engineering judgement, the longer than existing groyne would retain a wider beach. However, due to the closed sediment system, an initial beach nourishment followed by regular recharge would be required. Nonetheless, without detailed modelling, it was challenging to determine a profile of a likely retained beach and whether the beach alone, without raising the splash wall to the back of the promenade, would provide sufficient wave energy absorption during severe storm events. Moreover, the beach crest may lower during winter storms with consequent limited success in reducing the wave energy when most needed.

Another likely issue was increased sediment deposition at the mouth of the River Sid (to the east of the proposed long groyne), following south easterly storms leading to increase maintenance costs.

Due to time and budget constraints, in addition to technical and cost uncertainties, it was agreed by EDDC and the Sub-Group that this option would not be progressed forward.

2.2 Offshore breakwaters

The option of a series of offshore breakwaters was also evaluated and was found more feasible and likely to meet the requirements of the project.

Four breakwaters of different lengths were initially considered: two larger ones in front of the town beach and two smaller ones in front of East Beach.
The orientation of the breakwaters was discussed. Initial consideration was given to the possibility of implementing the breakwaters with an orientation similar to the existing ones, i.e. against south-westerly waves. However, this orientation would leave the Town front and East Beach exposed to south-easterly waves which could mobilise sediment and potentially overtop the sea wall and / or reach the base of the cliffs during sever events. A more parallel alignment to the coastline was considered more likely to produce the best results for protection against all wave climates. Moreover, the two existing breakwaters were found to provide significant protection to the western part of the town front from south westerly waves.

At East Beach, it was felt that the easternmost breakwater should be aligned more north- east to south-west to enhance the required protection in this area against south-easterly waves.

The existing splash wall at Frontage 6, by Port Royal, will likely need raising to provide adequate protection from south easterly waves. This element of the scheme has not been costed at this stage.

The existing beach will need to be renourished to the 1990 design as per BMP recommendation. This element of the scheme has not been assessed further nor costed in this exercise.

As offshore breakwaters were considered technically feasible and expected to meet the project requirements, a concept design was prepared to enable costing and evaluate the financial viability of the scheme.

Further refinement on the exact alignment and dimensions of the breakwaters require modelling which was outside the scope of this commission.

3 Site Conditions

The site conditions have been obtained from previous modelling work undertaken by RHDHV.

The waterfront was divided in frontages which helped to identify wave conditions for each of the structures. Figure 3-1 shows the frontage division used to identify the wave climate for each structure.

Figure 3-1: Frontage division for wave climate

3.1 Bathymetry

The bathymetry was extracted from the modelling exercise undertaken by RHDHV in 2018. Figure 3-2 shows an image of the model in which the contours of the bathy can be appreciated.

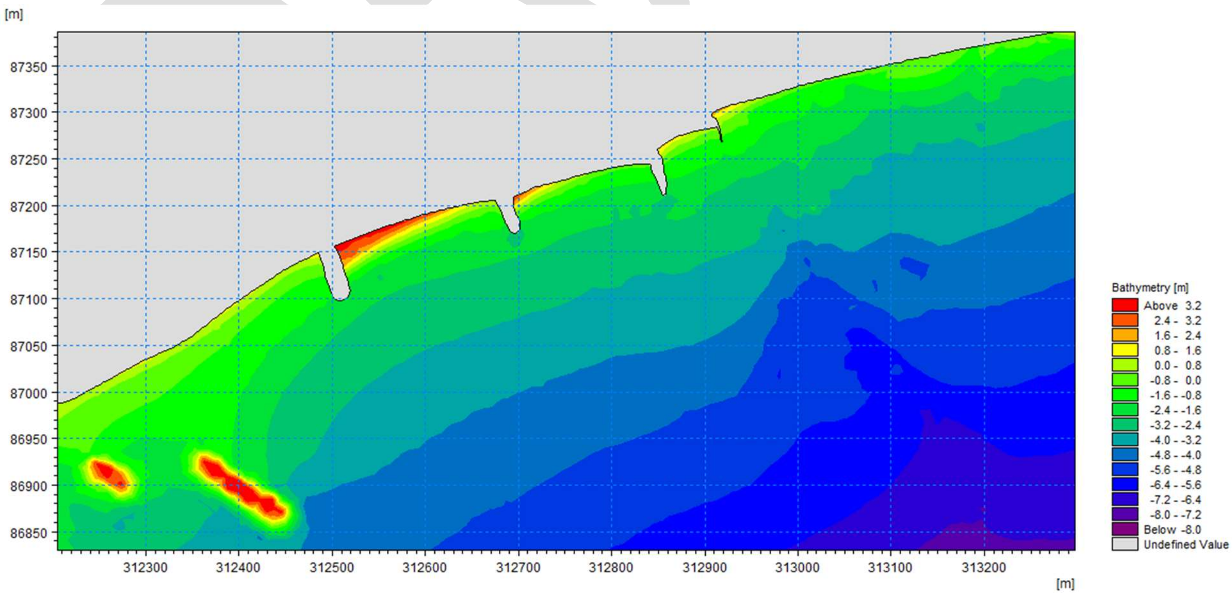

Figure 3-2: Bathymetry extract from numerical modelling at Sidmouth

3.2 Wave Conditions

3.2.1 Present Day

The present day wave conditions at the different frontages have been obtained from previous modelling work, which was summarised in report WATPB6525R001, from 17th August 2018. These wave conditions are shown in Table 3-1.

Table 3-1: Wave conditions at different frontages – South-westerly waves

Joint Probability Combination	Offshore Conditions			Nearshore Wave Height (Hs)							
	Wave Height (m)	Wave Period (Tp)	**Wind Speed (m/s)**	**Water Level (mODN)**	F1	F2	F3	F4	F5	F6	F7
1	11.15	14.14	25.89	2.41	4.11	4.09	4.29	4.42	4.59	4.16	4.11
2	10.33	13.66	24.84	2.47	4.09	4.07	4.27	4.40	4.56	4.14	4.09
3	9.81	13.34	24.15	2.55	4.10	4.08	4.27	4.40	4.56	4.14	4.10
4	9.26	13.00	23.41	2.61	4.09	4.07	4.25	4.38	4.54	4.14	4.09
5	8.54	12.53	22.40	2.69	4.06	4.03	4.21	4.34	4.49	4.11	4.06
6	8.00	12.17	21.60	2.75	4.05	4.00	4.17	4.30	4.44	4.09	4.05
7	7.45	11.79	20.79	2.81	4.01	3.96	4.11	4.24	4.37	4.06	4.01
8	6.74	11.27	19.66	2.91	3.92	3.87	4.01	4.13	4.24	3.97	3.92
9	6.20	10.86	18.77	2.98	3.83	3.79	3.89	3.99	4.09	3.87	3.83
10	5.66	10.44	17.85	3.05	3.70	3.63	3.67	3.75	3.83	3.69	3.70
11	4.95	9.84	16.55	3.15	3.43	3.30	3.28	3.33	3.40	3.33	3.43
12	4.63	9.56	15.94	3.18	3.25	3.10	3.07	3.11	3.17	3.12	3.25
13	4.41	9.35	15.50	3.21	3.11	2.95	2.92	2.97	3.02	2.97	3.11
14	3.87	8.84	14.39	3.29	2.70	2.56	2.53	2.56	2.60	2.57	2.70

The table above shows a series of offshore and nearshore (300m offshore) wave heights along the different frontages (F1-F7) for 14 joint probability cases between water levels and wave heights giving each of them a return period of 1 in 200 years. South westerly waves were found to cause the worst overtopping.

For this exercise, extreme water levels have been updated using the latest UKCP18 predictions.

A sensitivity test using several joint probability cases has been undertaken for the concept design.

3.2.2 Mid Term

Mid-term (2067) wave conditions have also been considered in the design. Only wave heights corresponding to Joint Probability Case 3 have been estimated as these produce the highest overtopping, see Table 3-2 below:

Table 3-2: Mid-term (2067) wave heights in meters along different frontages

F1	F2	F3	F4	F5	F6
4.27514	4.24892	4.43703	4.56727	4.7252	4.31969

3.2.3 Long Term

Similarly to the mid term case, the long term (2117) wave conditions for joint probability case 3 have been determined, see Table 3-3.

Table 3-3: Long-term (2117) wave heights in meters along different frontages

F1	F2	F3	F4	F5	F6
4.42	4.39	4.57	4.71	4.86	4.46

4 Design Approach

Concept design for three different locations have been undertaken:

- Layout 1: offshore structures
- Layout 2: nearshore structures
- Layout 3: intermediate structures

The three concepts have been conceived with the aim to meet the projects objectives, as described in Section 1.2.

Construction costs were estimated for the breakwaters based on two estimates received by EDDC for the construction of the existing breakwaters. Other elements of the scheme, such as beach nourishment, raising of the splash wall at discrete locations and risks were not included.

4.1 Layout 1 – Offshore Structures

Layout 1 comprises four breakwaters of different lengths, see Figure 4-1. Two breakwaters in front of the town beach have been located at ~200-250m from the shoreline and two in front of East Beach at ~135m and ~70m from the shoreline.

Initially, similar dimensions to the existing breakwater were chosen. The two breakwaters in front of the town beach have been assumed 135m and 140m long; the structure to the east of the River Sid in front of East Beach is 130m long and the easternmost is 75m long.

All proposed breakwaters are assumed parallel to the coastline, apart from the easternmost one in front of East Beach which is assumed at an angle providing best protection against south-easterly waves.

In agreement with the Option Appraisal (Appendix C) in the BMP, this alignment is considered most effective in reducing the wave climate reaching the beach and therefore reducing overtopping to acceptable rates. In front of East Beach, it is estimated that the chosen alignment would significantly reduce wave energy thus limiting the impact of winter storms on the toe of the cliffs.

Without detailed modelling (including sediment and physical modelling), it is not possible to confirm the effectiveness of the scheme, especially in limiting waves travelling through the gaps between the breakwaters. However, it is reasonable to assume that sufficient interference would be caused by the breakwaters to significantly reduce the wave energy reaching the shoreline.

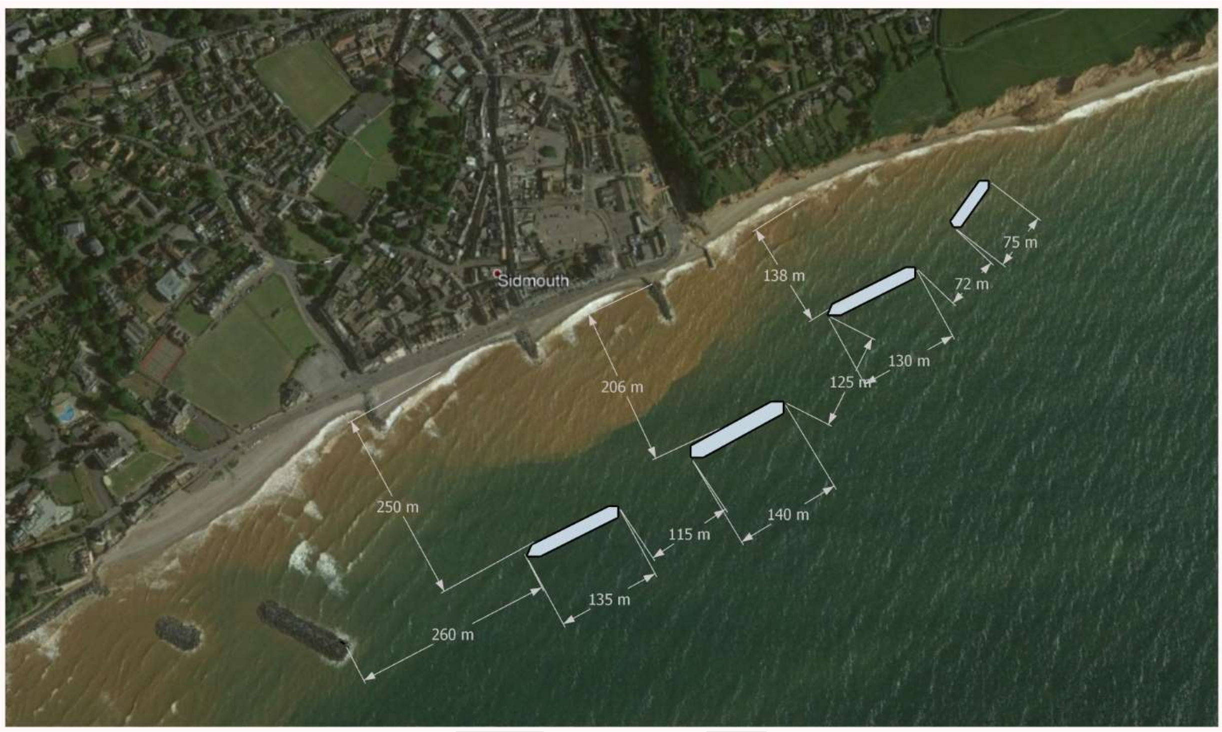

Figure 4-1: Layout 1 with offshore breakwaters

4.1.1 Type of Structures

<u>Town beach breakwaters:</u>
The breakwaters are designed as detached structures built at -4mOD, with a 2 layer 8t-12t rock armour with a 1:3 slope on the seaside and 1:2.5 slope on the leeside.

The core of the structures are designed as 1t-3t rock core and the whole structure is supported by a bedding layer formed of 100-1000kg rock laying directly on the seabed rock. The soft sediment is envisaged to be dredged, to allow for the structure to be supported by rock.

The dimensions of the crest of the breakwater have been determined considering overtopping and with the objective to achieve a transmitted wave passing over and through the structure which would allow to achieve acceptable overtopping rates at the town frontage. Further overtopping analysis at the town frontage showed that the assumed dimensions of 11.2m and crest level of 4.5mOD would dissipate the wave height sufficiently so that overtopping rates directly behind the breakwater would be ‘acceptable’ and not require a beach nourishment / design beach. Note: this is the case for the area directly behind the beach, assuming waves travel directly through the breakwater. Tests on wave overtopping between the breakwater structures, or wave transmission at different angles has not been undertaken. Although smaller structures could be potentially analysed at the same location in conjunction to the 1990 design beach nourishment, estimating the construction costs for these structures would provide a useful baseline for comparison to alternative potentially cheaper layouts.

The corner rocks at the crest and at the toe are to be within the upper 50% of the rock grading to increase stability at these locations, which are often the most vulnerable ones.

At the roundhead, the grading of the rock armour is increased to 10t-14t.

Figure 4-2 shows a sketch of the breakwaters in front of town beach.

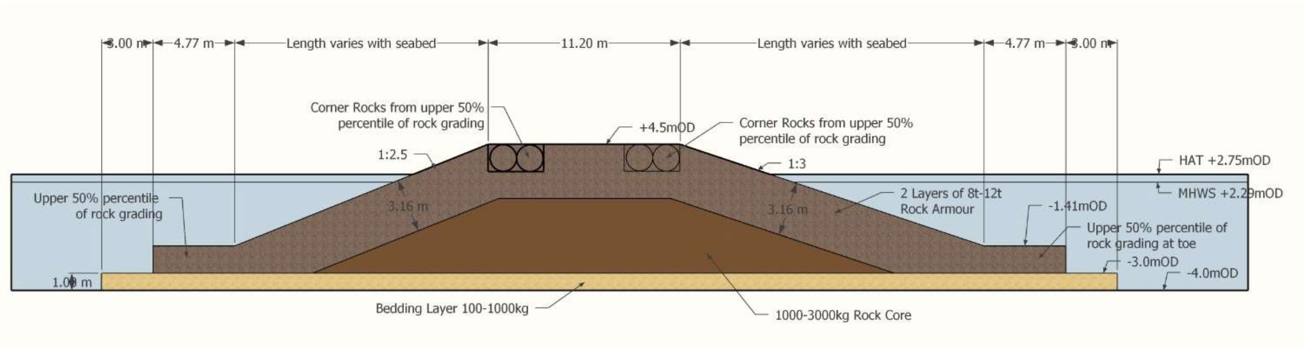

Figure 4-2: Sketch of breakwaters in front of town beach

East beach breakwaters:

The breakwaters on East beach have a similar structure as those on the town beach. The main difference is that the armour size is designed as a 2-layer 6t-10t rock armour with a 1:3 slope on the seaside and 1:2.5 slope on the leeside.

The core of the structures has 1t-3t rock and the whole structure is supported by a bedding layer formed by 100-1000kg rocks laying directly on the seabed rock. The soft sediment is envisaged to be dredged, to allow for the structure to be supported by rock.

The dimensions of the breakwater crests have been determined considering overtopping and the reduction in energy transmission from waves. It has been determined that a crest width of 11.2m and a crest level of 4.5mOD would provide a substantial reduction in wave height behind the structures. This reduction, in combination with the beach recharge at East Beach, was considered to be sufficient to provide enough protection to the base of the cliffs for the medium to long term. However, further detailed analysis is required to confirm the assumptions at design stage.

The corner rocks at the crest and at the toe are within the upper 50% of the rock grading to increase stability in these points, which are often the most vulnerable in the structure.

At the roundhead, the grading of the rock armour is increased to 8t-12t.

The easternmost breakwater, closer to the shore, is founded in shallower waters, at -2.5mOD instead of -3mOD for the one closer to the mouth of the River Sid.

Figure 4-2 shows a sketch of the longer breakwater at East Beach

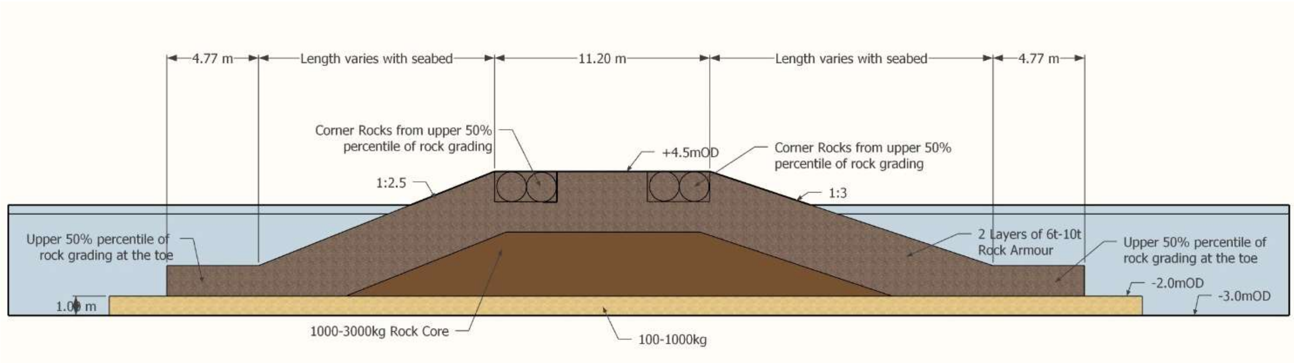

Figure 4-3: Sketch of breakwater in front of East Beach

4.1.2 Cost Evaluation

Following the design of the breakwaters, an estimate of construction costs of the structures only was undertaken.

The costing exercise showed total construction cost for the breakwaters of ~£18.5M. This did not include specific construction risks. The two main breakwaters in front of the town beach were estimated at £6.0M each, the largest breakwater in front of East Beach at ~£4.5M and the smallest one at ~£2.0M.

4.1.3 Conclusions

Although the exact layout and dimensions would need further refinement at outline and detailed design stages, Layout 1 was considered to be technically viable and to meet the requirements of the project in terms of overtopping of the waterfront and protection to the cliffs at East Beach for both the present day and the mid-term, while maintaining the overall structural stability of the structures.

However, given the existing budget constraints, the scheme was not economically viable. The possibility to implement smaller structures by moving them closer to the shore were investigated.

4.2 Layout 2 – Nearshore Structures

The design of Layout 2 is conceived by moving the structures much closer to shore to evaluate the potential construction cost savings whilst maintaining sufficient wave energy absorption to limit wave overtopping / cliff erosion. These structures are considered in combination with shallow tombolo / salient at the beach.

The structures are located ~70m from the shoreline both at the town beach and at East Beach.

Shorter breakwaters have been designed in comparison to Layout 1, as the closer proximity to the shoreline allows to take more advantage of the existing groynes and the presence of shallow tombolos to the lee of the structures. The length of the breakwaters in front of the town beach is ~100m, and at East beach ~90m and ~60m respectively.

The overall layout can be seen in Figure 4-4.

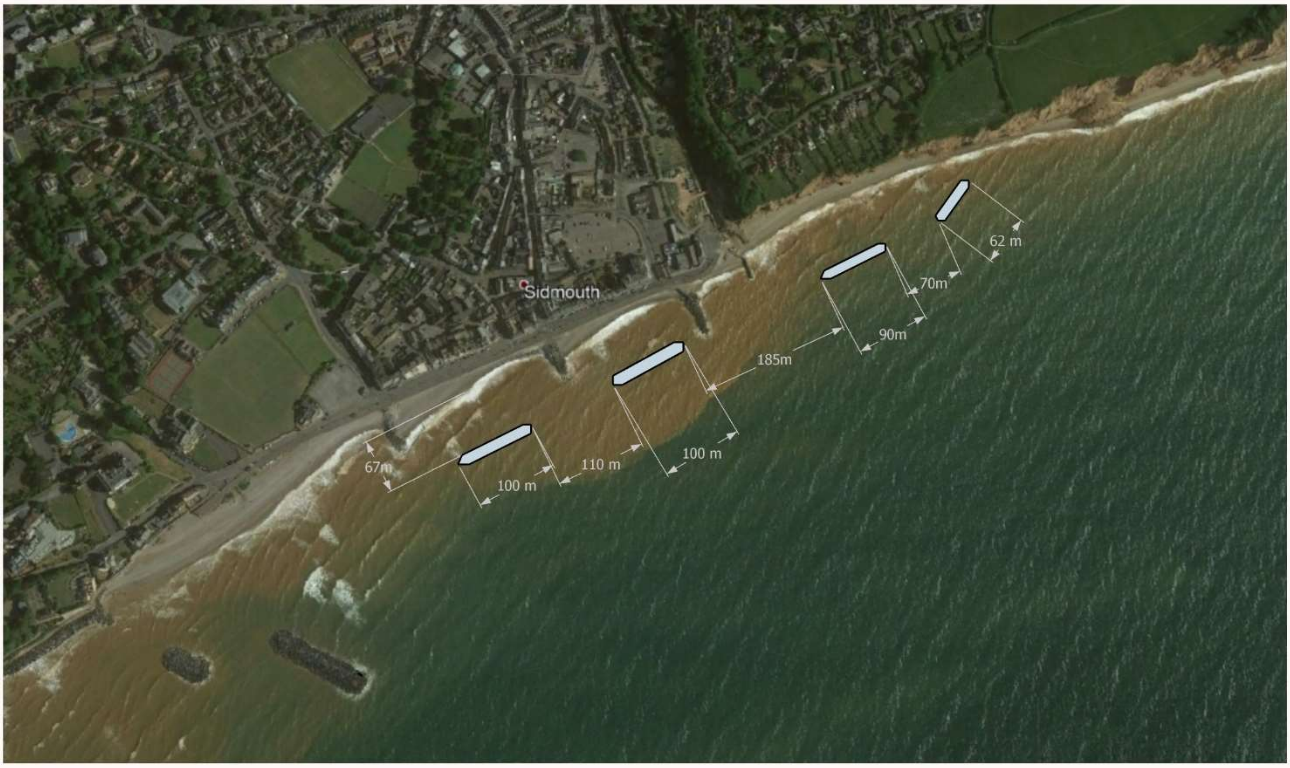

Figure 4-4: Layout 2 with nearshore breakwaters

4.2.1 Type of Structures

Town beach breakwaters:

The closer proximity to the shore and the limited water depth made the structure to be designed as a single rock type with no core but all rock armour. The depth limited wave heights allow a reduction in the armour grading at the trunk to 6t-10t in comparison to the 8t-12t of the offshore location.

The armour slope is still designed as 1:3 on the seaside and 1:2.5 on the leeside.

The structure is still envisaged to be founded on a bedding layer of 100-1000kg, which would be directly laying on rocky seabed after dredging the soft sediment.

The dimensions of the crest of the breakwater have been determined considering greater overtopping and wave transmission rates than for Layout 1. However, these structures would work in combination with shallow tomobolo / salient features to their lee which would contribute to the dissipation of the surplus energy. Costs to create these features would need to be accounted for in the overall scheme design, as although these would be created naturally by the breakwaters, to be effective the scheme needs to rely on their presence since implementation. In this layout, the crest width is decreased to 9.8m and the crest height to +3.75mOD, reducing construction costs.

The corner rocks at the crest and at the toe are within the upper 50% of the rock grading to increase stability in these points, which are often the most vulnerable locations in the structure.

At the roundhead, the grading of the rock armour is increased to 8t-12t.

Figure 4-5 shows a sketch of the breakwaters in front of the town beach.

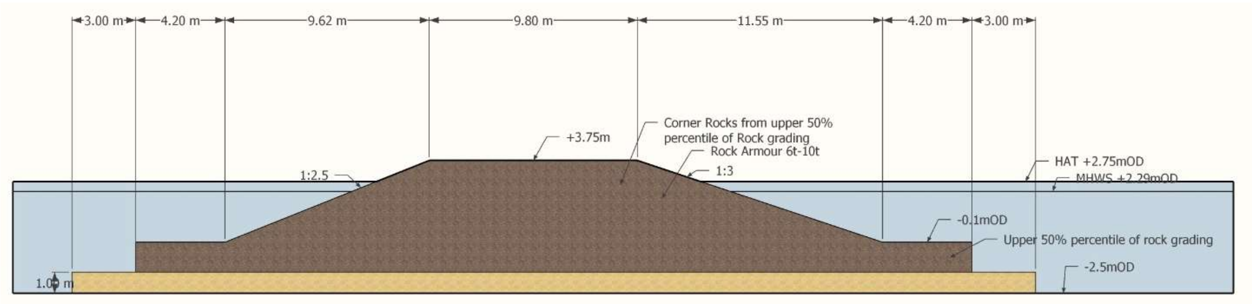

Figure 4-5: Sketch of Breakwater in front of town beach

East beach breakwaters:

The breakwaters at East beach have the same structure as those on town beach. The only difference is the foundation depth for the smallest and easternmost breakwater which is at -2.0mOD.

Figure 4-6 shows a sketch of the most eastern breakwater on East Beach.

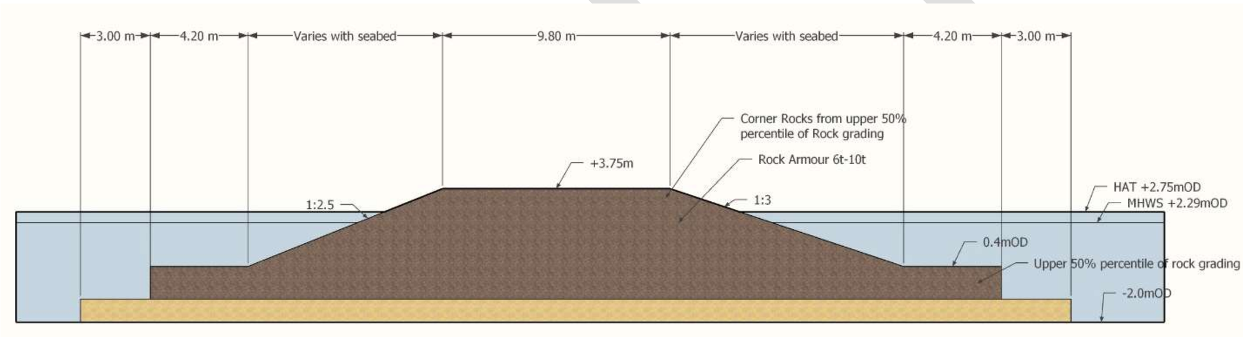

Figure 4-6: Sketch of eastern breakwater in front of East Beach

4.2.2 Cost Evaluation

The cost evaluation exercise for Layout 2 scheme showed a significant reduction in construction costs for the breakwaters compared to Layout 1, due to the shallower location of the breakwaters (~1.5m shallower than Layout 1) and smaller dimensions.

The overall cost for the construction of the breakwaters as per Layout 2 is ~£8.9M. The cost of the two breakwaters in front of town beach are ~£2.6M each. On East Beach, the western breakwater is ~£2.3M and the smallest breakwater is ~£1.4M. This estimate does not include any specific construction risks.

4.2.3 Conclusions

Layout 2 is a scheme that would technically meet the requirements of the project with an appropriate beach renourishment and potential raising of the splash wall at Frontage 6. Numerical modelling is required to confirm layout and sediment transport in the long term. Furthermore, this scheme reduces construction costs to about 50% compared to Layout 1. However, the close proximity of the breakwaters to the existing beach coupled with the formation of tombolos would significantly reduce the swimming area and potentially neglect the amenity value of the waterfront. In addition, the additional costs for beach nourishment, raising of the splash wall and risks are estimated to be greater than the available funding.

The possibility of relocation structures of similar dimensions further offshore, between layouts 1 and 2 was investigated. This would maintain the lower construction costs but provide the much-needed recreational ground for residents and tourists.

4.3 Layout 3 – 150m Offshore Structures

Layout 3 is a modification of Layout 2 in which structure of similar size are relocated slightly offshore, 150m from the shoreline. The aim of this layout is to maintain lower project costs whilst achieving the remaining project objectives. Therefore, whilst the dimensions of the structures are the same as in Layout 2, these are located further offshore and 0.5m deeper.

A sketch of the layout is shown in Figure 4-7.

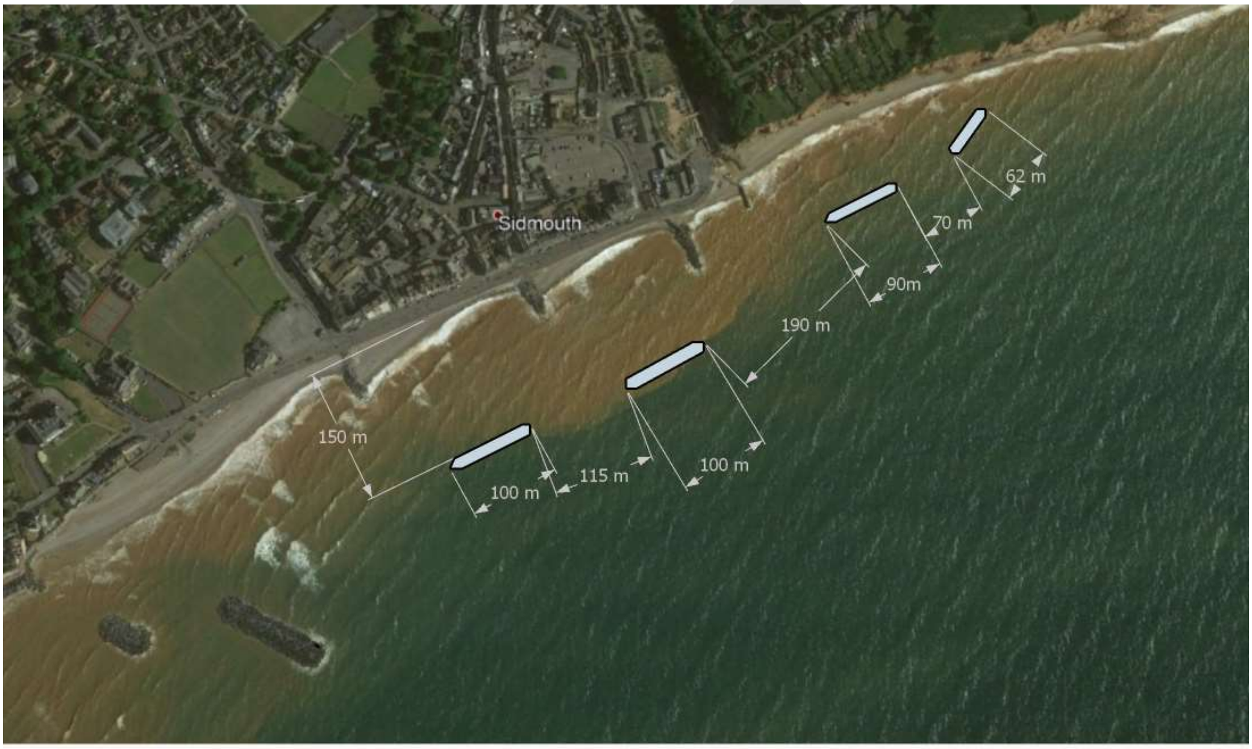

Figure 4-7: Sketch of Layout 3 with breakwaters located 150m offshore.

4.3.1 Type of Structures

Town beach breakwaters:
At 150m from the shoreline, at a depth of -3.0mOD, incident wave heights during a 200 year storm event are not depth limited and therefore a bigger grading of rock armour of 8t-12t is needed to ensure stability.

The slope of the structure on the seaside is 1:3 whereas on the leeside is 1:2.5.

The same foundation detail of Layout 2, comprising a bedding layer of 100-1000kg after dredging of the soft sediment is envisaged.

With regards to overtopping and wave transmission, the relocation of the breakwaters into deeper waters without increasing the height of the structure translates into a smaller freeboard during a storm. Therefore, the amount of overtopping and energy transmission over the structure is estimated to be greater. However, similar to Layout 2, the structures are conceived to work in combination with a renourished beach on their leeside forming shallow tombolos adequate to reduce overtopping over the waterfront to acceptable limits. It is estimated that the assumed dimensions shown in Figure 4-8 are appropriate and stable to withstand storms with present day values. However, by mid-term (2067), the structures are likely to experience damage both at the seaside and leeside.

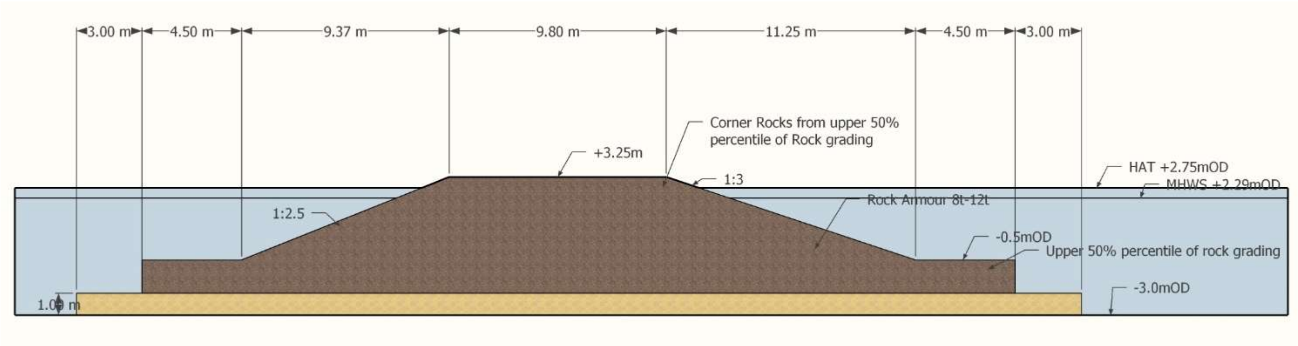

Figure 4-8: Sketch of town beach breakwaters according to Layout 3

<u>East beach breakwaters:</u>
The breakwaters at East beach have the same structure and location as those in Layout 2.

Figure 4-9 shows a sketch of the westernmost breakwaters on East Beach.

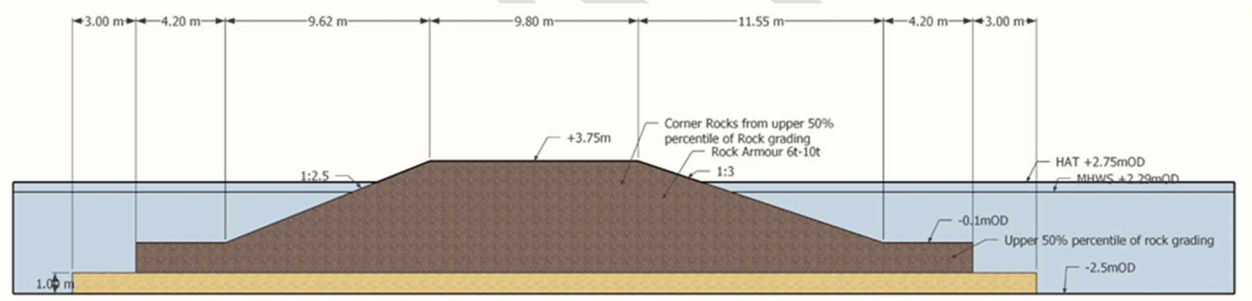

Figure 4-9: Sketch of westernmost breakwater at East Beach

Figure 4-10 shows a sketch of the easternmost breakwater at East Beach.

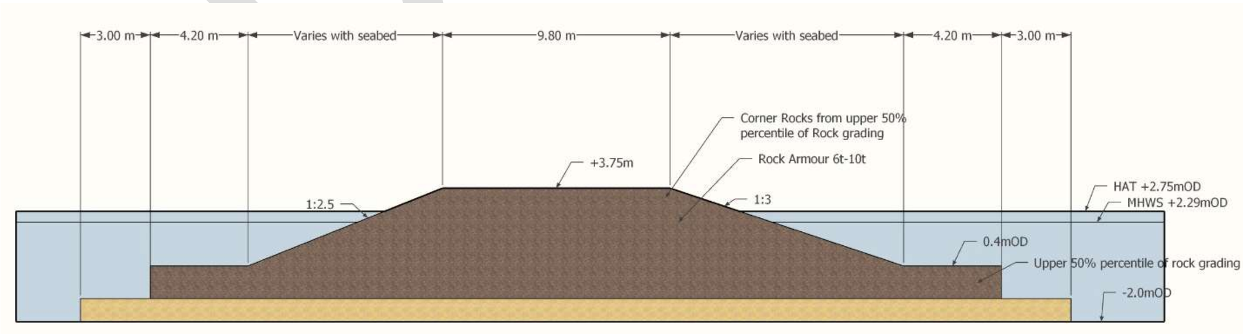

Figure 4-10: Sketch of eastern breakwater in front of East Beach

4.3.2 Cost Evaluation

Construction costs for the breakwaters in Layout 3 are the same as for Layout 2, as the same dimensions and lengths of the breakwaters have been maintained: total costs of ~£8.9M, comprising of ~£2.6M each for the breakwaters at the town beach and £2.3M and £1.4M respectively for the structures at East Beach. No specific construction risks were included.

4.3.3 Conclusions

The scheme comprising the four breakwaters at the location shown in Layout 3 and beach nourishment is considered to be effective to meet the requirements of the project, including maintaining the amenity value of the beach in front of the town.

The stability of the structure is considered to be adequate for the present day condition but the concept design assessment showed that it would be likely to experience damage both at the seaside and potentially at the -leeside in the mid-term (2067). However, numerical modelling is required to confirm the performance of the structures and to determine the required dimensions of the renourished beach. Also, it would be necessary to undertake sediment transport modelling to ensure that the structures are effective in preventing washing out of the sediment and to demonstrate that the beach would be stable in the short to long term.

Despite construction costs being reduced, it is estimated that the overall project costs, when considering beach nourishment, raised splash wall and risks would be higher than the available funding. Therefore, this option although technically viable, would not meet the overall economic requirements of the project and should not be explored further.

5 Conclusions and Next Steps

RHDHV have provided conclusions on the three schemes and layouts considered for the coastal scheme at Sidmouth. These conclusions summarised technical issues and viability, cost estimates and other considerations such as the amenity value of the beaches. It should be noted that these conceptual layouts have been developed based on engineering judgement and high-level calculations, without the use of numerical modelling to confirm the layout, including the development of the wave climate and sediment within the gaps between the structures and general behaviour of the sediment.

The assessment of the first layout was undertaken considering offshore structures in front of town beach and the 1990 beach profile. Although shallow tombolos may still form in the medium to long term, these are not to be relayed upon in the short term to absorb surplus wave energy through and over the breakwaters. Therefore, bigger structures have been designed to ensure sufficient wave energy is absorbed offshore.

Layouts 2 and 3 have been developed in combination with tombolos / salients established at the same time as constructing the breakwaters to their lee and subsequently maintained by the structures. These features would act in combination to the structures in absorbing wave energy and thus protecting the town frontage and the toe of the cliffs. In addition, the whole frontage would be renourished as per the 1990 profile to provide the required level of flood and erosion protection. The structures height and crest width are reduced allowing a higher overtopping rate, with the view that the beach and features to the lee would dissipate a larger amount of energy.

Whilst Layout 3 was considered the preferred option, overall construction costs would be higher than the available funding and therefore it was recommended not to explore it further in isolation.

In order to refine Layout 3 numerical modelling (wave, overtopping and sediment transport) would be required to assist with the following tasks:

- Confirm wave height values at the leeside of the structures and in front of the beaches, as a result of wave penetration and wave transmission through the structures;
- Confirm wave height transmission in the gaps between the structures;
- Determine the required dimensions for the renourished beach to achieve acceptable overtopping rates at the waterfront;
- Investigate alignment of the breakwaters to provide best reduction of wave energy for most wave climates;
- Investigate sediment transport after the implementation of the structures;
- Update incoming wave conditions to allow an accurate evaluation of the stability of the structures in the mid-long term during detailed design.

Error! Reference source not found. shows a summary of the construction costs for the breakwaters for the different layouts:

Table 5-1 - Breakwaters construction costs for different layouts

	Layout 1	Layout 2	Layout 3
Breakwater 1 – town frontage	£6.0M	£2.6M	£2.6M
Breakwater 2 – town frontage	£6.0M	£2.6M	£2.6M
Breakwater 3 – East Beach west side	£4.5M	£2.3M	£2.3M
Breakwater 4 – East Beach east side	£2.0M	£1.4M	£1.4M
Total	**£18.5M**	**£8.9M**	**£8.9M**

<u>**Hybrid Solution**</u>

Following a meeting with EDDC and the Sub-Group on 02/09/2021, a hybrid solution was suggested comprising the following elements:

- Beach nourishment along the town beach frontage to the 1990 scheme profile, as per original OBC;
- Raised splash wall at Frontage 6, as per original OBC;
- Raising existing splash wall by 100mm along the promenade with new foundations enabling raising of the splash wall further when overtopping rates become unacceptable in the mid to long term (to be refined at detailed design stage);
- New long groyne and beach nourishment at East Beach, as per original OBC;
- Use surplus funding, following updated PF calculator, to construct one or two breakwaters in front of the Town beach. Increased risk pot and other elements to be consider when assessing available funding.

This option would reduce the need for maintenance / recharging of the beach by EDDC, by introducing offshore structures which would ensure a more stable beach at the town front.

Further investigation is needed to assess the full viability of this option, including but not limited to revised economic analysis, environmental assessment, visual impact and risk register.