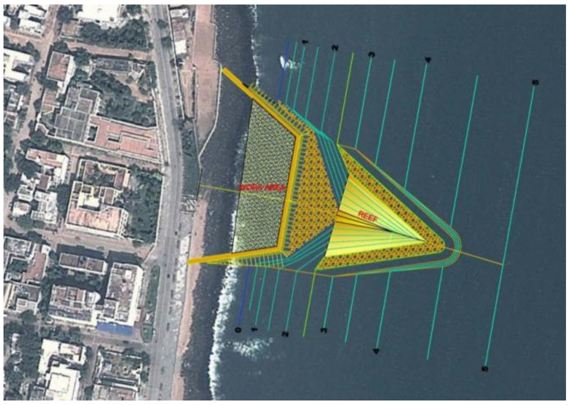
Detailed Design Report on Pondicherry Beach Restoration Project

Part II



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SUMMARY OF METHODOLOGIES

This report consider at least 5 construction methods for each reef, i.e. rocks, concrete, geotextiles, steel and any other alternatives. Moreover, the wedge reef is a multiple project with 6 separate components, i.e. the work area and retaining walls, the segments between the reef and work area, the underlying rock platform, the scour protection, the higher wave zones identified by computer modelling and the wedge itself. Each of these require different rock sizes, while the wedge is a component that could be made of geobags, concrete, steel or even HDPE. The wedge is an independent component which weighs thousands of tonnes and so methods to put this in place had to be devised without any precedent in India or globally.

The southern reef also required an assessment of multiple options for construction and we have considered rocks, geotextiles and steel construction. The nourishment has been designed, layed out and advice given about volumes and placement.

The wedge Reef

There are six (6) main components on the wedge reef. These are:

- The wedge itself which is a triangular shape some 60 m across the base and 60 m along the spine. The elevation is 2.5 m
- The materials which lie under the wedge to create a horizontal platform
- The material to be placed at depth around the wedge to act as toe scour
- The materials to be placed in the zone between the wedge and the work area
- The construction of the walls for the work area
- The protection of zones which were identified by detailed computer modelling to be prone to high waves

The wedge

Here we consider just the wedge itself, i.e. the 60x60 m triangular element resting on a rock bed.

Geotextiles were initially considered for the wedge component. However, these materials were not suitable over a rock base. In the absence of the rocks, there were concerns about the reef sinking. In addition, the number of geobags was too large for the budget. Finally, the construction time to fill the geobags in situ was found to exceed 2 years, given the small number of suitable days for filling when the waves are small enough. These factors eliminated the geotextile option.

The next alternative was rocks. However, the offshore tip of the wedge would be at Chart Datum and computer modelling showed that in big waves, the wedge would be exposed in the trough. This meant that very large rocks with tetrapods would be needed to protect the tip of the wedge. It would also need to be redesigned with a broad curved nose, as the fine tip of the structure would not be stable under heavy wave attack. The design wave exceeded 4 m and the front of the reef is steep so that the breakers were expected to plunge and thereby exert more force and the stability was challenged. Such large rocks are not readily available in Pondicherry which meant that the cost rose substantially. The use of rocks would also eliminate any possibility of adding amenity such as surf riding to the structure. These factors suggested that rocks were not suitable, although the option has been kept in reserve.

Initial assessments were done with formed concrete. The weight was high and so we examined the possibility of using a lighter material such as thick HDPE sheets. Unfortunately, the sheets manufactured in India proved to be too narrow which would have required too many joins. The joins are a weakness and methods were then designed to make stronger joins using stainless steel plates bolted together. However, the cost proved untenable and concerns remained about the durability of the HDPE material, as it hadn't been used in the surf zone before. This option was therefore deemed unsuitable.

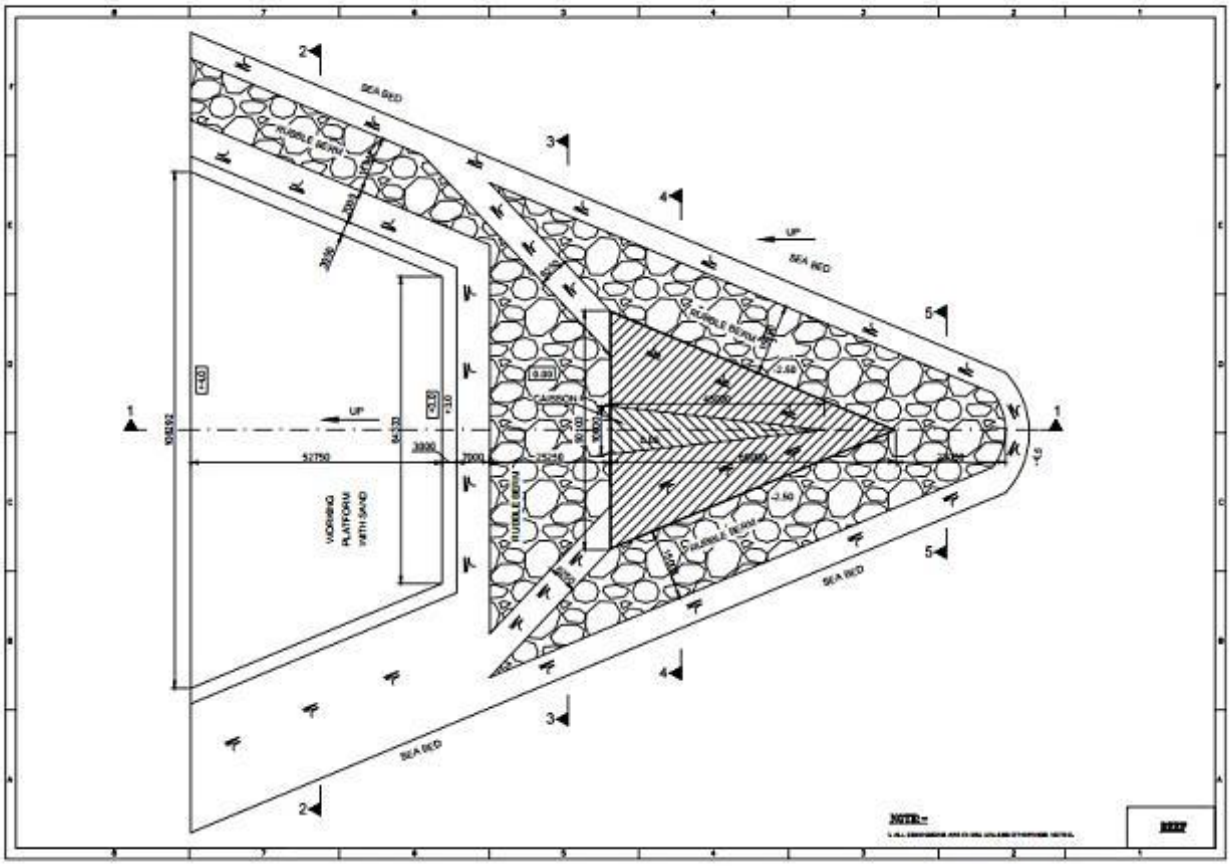
Returning to formed concrete with voids, detailed designs were undertaken. The overall weight of the reef was found to be greater than 5000 tonnes. This was untenable and moreover the object was too big to handle by any means other than manufacturing near the sea, towed into the water and floating to the site. However, with the preferred wall thicknesses of 300 mm the reef was too heavy to float. Concepts of additional flotation were rejected due to the large volume of floats required. There were also concerns that the reef would have an untenable draft in the shallow waters around the Port of Pondicherry. Dr. Chandramohan was able to reduce the wall thicknesses to 150 mm by using more internal reinforcement which finally allowed the wedge to float. However, a dry dock needed to be constructed to get the wedge off land, which proved to be more expensive than the reef itself and the whole operation was beyond budget.

The only tenable alternative for a concrete structure was to break the wedge into numerous smaller components, but there were concerns that joining these heavy elements on site might be difficult, particularly over an uneven rock base. If the joins are not complete, the wedge may prove to be dangerous and the smaller rocks under the wedge may be lost.

Further, a large crane which can lift 100 -200 t is required to lift the caisson sections. The crane requires access to the crest of the reef and this could in turn dictate the elevation and width of the crest. In short, the construction will be difficult and will require different construction methods and plant. One suggestion was to use an overhead crane system erected on piles hammered in on either side of the reef, but this would add substantially to the cost which is not in the present budget. This option of using multiple units remains viable and a method has been described which may be appealing to the TCEC and the contractor.

During this process, both Mr Steinhobel and Dr Chandramohan considered the steel option. Mr Steinhobel felt that steel was ideal but expressed concerns about rusting. Solutions were obtained including the use of sacrificial anodes and the selection of best quality steel with welds using similar metal. All the welds would be on the inside to reduce exposure to oxygenated seawater after the reef was fully sealed once deployed. A steel design was undertaken by Dr Chandramohan which is reported in this document. The cost rose with the use of thicker steel (25 mm) and more internal supports as the design proceeded. However, the steel wedge remained within budget and is a viable option. In particularly, it can be made in a single piece using workers who have experience in the construction of steel vessels. In essence, the wedge is like the upside down bow of a ship. Another benefit of the steel is that the base is not fragile and it can be put onto a more uneven rock base than concrete, which is more easily spilt or broken. The steel will float easily with a draft of 1 m. It can be deployed by simply allowing water inside, it doesn't require filling with sand and so it remains removable by injecting air to re-float it. The steel reef can be deployed in one day, once all site preparations are completed.

The only other alternative that remained viable was the numerous concrete sections. The reef would be built with caissons placed along the centreline, and then pieces small enough for a crane would be placed along both sides of the spine to form the shape. As noted above, the joining of the pieces could be done using an interlocking (male / female) segment, but the wave conditions on site and the bed level make this option more time consuming and while the segments are being placed, the waves could disrupt the rock platform below. Overall, we believe that this option remains viable and the views of the contractor responsible for construction would be valuable.



The Wedge Reef

The materials which lie under the wedge

If the wedge is made of concrete, the base thickness will be only about 150 mm. In this case, the underlying rocks will need to have a smooth surface to prevent rocks breaking and penetrating the base. Of course, if 25 mm steel is used, then these concerns are much less.

We have designed the base under the wedge to consist of two layers. The first layer will be fine stone (0.5 m) to form a bed to inhibit sinkage. Larger rocks (1.5 t) will then be placed on top, one by one to form a smooth surface similar to a tessilated pavement. These larger rocks need to be stable until the wedge is placed on top. If the wedge is broken into pieces, each section of bed could be prepared just for the next piece which reduces the time that the prepared bed is exposed to waves. This element will prove to be difficult in the "brown" waters off Pondicherry and with a wave climate that is small but very consistent. Costing this element is difficult to achieve without putting the works out to tender. However, we have taken considerable advice and provide the best possible costs at this time. A road out to the reef would be built first from the work area, and then cranes would place the rocks. Some parts may require an excavator also.

The materials needed for toe scour

The toe of the reef lies in depths deeper than -2.5 m and so these materials can be rocks of just 1.52 t. The model shows a broad zone of scour at times, and so we had to extend the width of the toe scour platform to allow for subsidence of the edges as erosion occurs.

The materials to be placed in the zone between the reef and the work area

Some members of TCEC have requested that geotextiles be used in this zone. Others have preferred rocks. Thus, we have examined both options. Concerns about the geotextiles include the cost and the reliability, particularly in a zone which will be open to the public at beach level. Our bag layout showed that some 27 geobags each 20 m long would be needed. The cost was much higher than rocks. One benefit is that the zone on the north end of this region is prone to high waves and may need large rocks, if geotextiles are not adopted. There are examples in India where the geotubes have not proved reliable and they will require a different contractor with geobag experience to properly fill them, which complicates the tendering for the project.

The walls around the work area

These sorts of walls are not novel in India. They will need to be constructed with rocks of 0.5 t. However, the north-east corner protrudes into natural depths of around -2.5 m and is directly exposed to heavy wave attack during the NE monsoon. Larger rocks are needed on this corner.

The walls will be constructed, lined to landward with geotextiles and the region will be filled with sand.

Southern Reef

The southern reef is 200 m long and will be placed in 4 m depth. The crest height is 1 m above Chart Datum making the structure 5 m tall. It can be constructed from:

- Geotextiles in a 4 layer structure, after allowance for sinkage
- Rocks and tetrapods, like a common breakwater in India
- Concrete caissons which are floated into position and filled with sand
- Steel caissons which are floated into position, filled with sand and then buried under rocks for stability and to induce wave breaking. The rocks would be at least 1 m

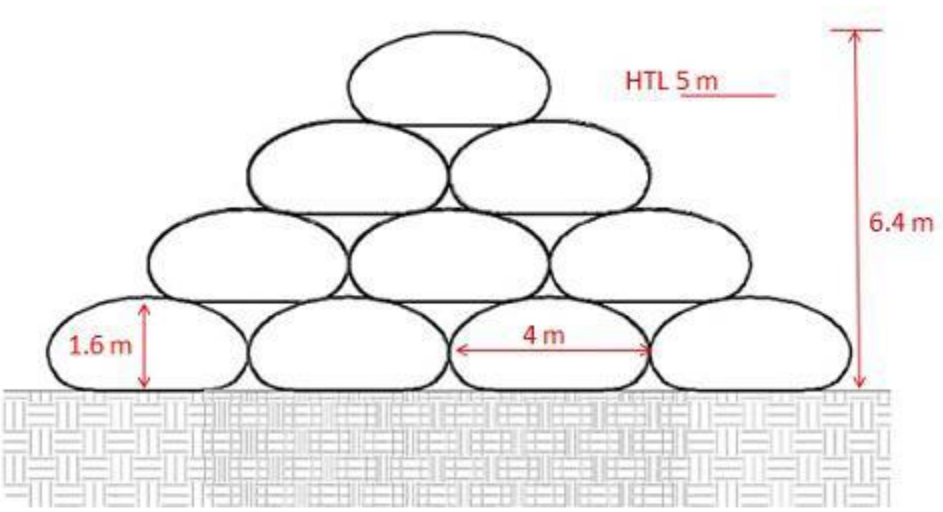
below the surface and so smaller rocks would be required for this option and no tetrapods. A novel design has been developed from steel.

No other options came to light.

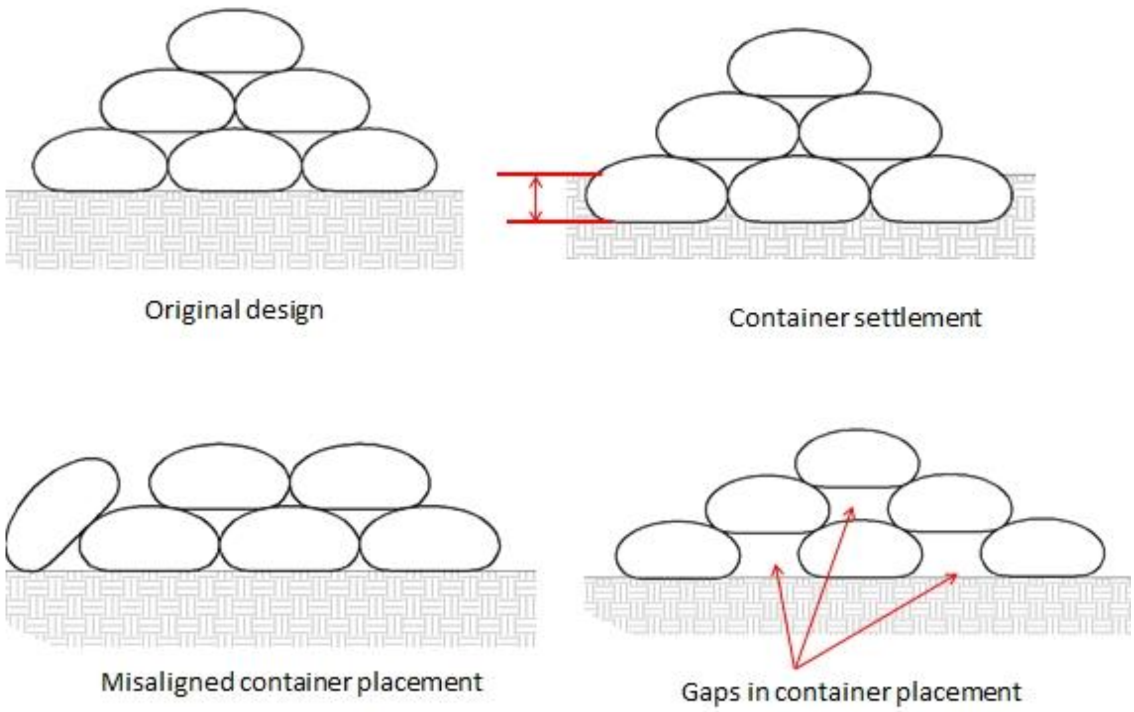
Geotextiles

We have designed a geotextile reef which consists of 4 layers of geobags, 11.5 to 28 m long and 1.6 m high and 4 m wide (noting that the lower bags will be compressed by the weight above). A total of 194 geobags will be needed and sand volume required is 13,000 cu m. Several concerns were identified. First, the geobags are narrow and prone to sinkage. We estimate that the total sinkage may be as much as 2 m and so the bottom layer is sacrificial. Secondly, a 4-layer structure needs a broad base to hold the large bags at the top as these are directly attacked by waves which could be as high as 6-7 m after shoaling in the 5 m water depth at high tide. They will be plunging on the steep reef face. Thus, large bags of at least 11.5 m length are needed in the top layer. If the bags are not filled under full compression, the waves cause the sand in the bag to migrate to the back of the bag which can lead to flapping of the front and eventual destruction of the geobag.

Experience with filling indicates that it will take at least 1.5 days per bag for placing, filling, topping up and capping. So the total construction time will be a minimum of 10 months. However, there will be numerous days when the waves are too large to operate and so the construction time is estimated to be at least 2 years. Finally, many bags placed in heavy wave zones have had their caps ripped off by the waves which causes the bags to empty and disintegrate. The caps must be placed over the “trunks” or the inlets needed for insertion of the dredge pipe into the bag for filling. They are difficult to secure because the plastic bolts and ties are into geotextile which is not strong enough to hold the caps under heavy waves. The geotextile reef is also beyond the budget.



4-layer geotextile reef



Geotextile mega container placement failures

Rocks and tetrapods

Because the reef is sitting in depths of 5 m at high tide, the wave climate is very severe. An allowance of 6-7 m waves is required. Thus, the rocks required near the crest are 6-8 t, and they will need to be protected with tetrapods for added security. To get the stability, the front gradient will need to be at least 1:4. With the high cost of large rocks and the broad base and volume of the reef, the rock structure is beyond the budget. Some observers have expressed the view that the rock and tetrapod reef will be unsightly.

Concrete caissons

Concrete caissons remain viable and they could be floated into position. Their size may be minimised by placing rocks at the front and back of the reef. However, concrete caissons were rejected as unsuitable by the TCEC at the end of Task 1 and so no further work on this option was undertaken.

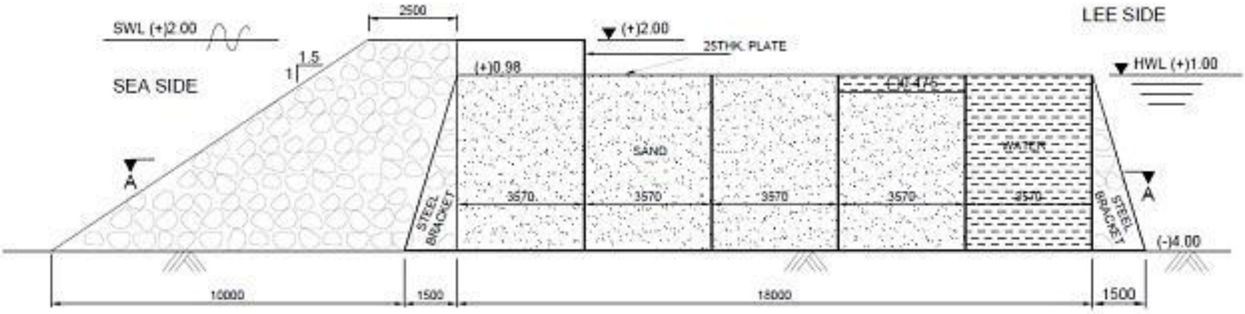
Steel caissons

As for the wedge, the steel caissons are more buoyant than concrete and they have intrinsic strength. We have produced a novel design to reduce cost and visual impact. Essentially, the structures consist of a cuboid (caisson) base to be filled with sand. This would remain underwater. To reach the 1 m level above CD, a reinforced wall will be built from the base. This narrow wall will have minor visual impact but will act to disrupt the waves. For better stability at lower cost, rocks would be placed at the front and back of the wall, over the base and in the lee of the reef to stop it moving shorewards under wave attack. The rocks would remain underwater around 1-2 m depth, to reduce the size required. To dissipate wave energy and to minimse over-toppng and wave reflection from the vertical impermeable steel sheet, tetrapods can be used in the armour layer in front of the caisson. The rough surface they create is highly efficient for dissipating wave energy.

The steel caissons float with low draft and so they are suitable for towing into position. They will be stable under their own weight in small waves, which will give time for filling and rock placement.

At this stage, our initial stability analysis suggests a caisson of 6 m tall, 8.5 m wide and 25 m long is stable. The cost is within budget and so discussions are continuing among the engineers to optimise the size of the steel structure against the stability demands, while potentially reduced the height of the caisson and replacing the top 1-2 m with a wall. The concept is to fill the caisson with sand and rock to increase stability. Submerged tetrapods will be of assistance on the front side of the reef to dissipate wave energy.

The steel structures would be each 25 m long and the reef would be made in sections of this length with 3 m gaps. The gaps are designed to allow over-topping water to flow back out to sea, rather than scouring the sand in the lee of the reef and disrupting the shore protection. The gaps would be filled with rocks to mid-depth to prevent scour. The structure would be 6 m tall overall, to allow for 1 m of sinkage.



South Reef : steel caisson (to be replaced with the new design - 6 x 8.5 x 25 m caisson)

Construction considerations

The requirement is to bring many alternatives with discussion, consideration of the detailed construction methods, detailed costs and the bill of quantities.

- Detailed design of identified eco-friendly protection measure
- Detailed engineering
- Stability analysis
- Drawings
- Bill of Quantities
- Costing

As in all engineering projects, the multiple options take considerable effort to bring to a stage where a detailed costing and Bill of Quantities can be done.

Summary of Cost Estimate

Reef	Amount (INR)	Amount (US$)	Construction period	Remarks
North Wedge Reef				
Wedge reef with rock base and single 150 mm thick slab concrete caisson	32.36 Cr	4.89 Million	6 months	High cost, caisson floating not feasible
Wedge reef with rock base and 300 mm thick slab concrete caisson in 10 sections	17.38 Cr	2.59 Million *note	6 months	Construction difficulty, heavy equipment requirements
Wedge reef with rock base and 25 mm thick steel caisson	16.95 Cr	2.53 Million	4 months	Easier construction
South Reef				
4-layer geotextile reef	21.03 Cr	3.14 Million	2-3 years	Long construction period and risks associated with construction and material damage
16 mm steel caisson reef with scour protection rocks	19.87 Cr	3.00 Million	3 months	Construction can be completed in one season

- *note We have not included the cost of an overhead crane system for placement of the concrete caisson sections. We estimate this may add substantially to the budget for the wedge reef with 10 sections. This cost has not been included in the Table. The crane is not needed for the steel caissons.

List of Drawings

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1 Introduction

1.1 Background

To develop site specific and environmental-friendly coastal protection design options for beach restoration at Puducherry and to implement the suitable coastal protection option, the work is undertaken in 3 phases:

Task-1: Feasibility Studies of Design Alternative (Coastal processes investigations, numerical model studies, site specific design options)

Task-2: Detailed Design and Construction Methodology (design finalization, engineering drawings, bill of quantities, tender documents)

Task-3: Project Implementation and Construction Management Support

Task 1 systematically assessed more than 30 options for the coastal protection at Puducherry, and eventually focused on four solutions with beach nourishment. Two of the options were offshore and the third option was hybrid with a reef offshore and a structure on the beach. The project's Techno Commercial Evaluation Committee (TCEC) preferred a new fourth option, which is a nearshore wedge reef with the crest at Chart Datum and the reef spanning the zone between the 0 m and the 4-5 m depth isobaths. Consequently, the nearshore wedge reef was carried through to Task 2 for detailed design, at the discretion of the TCEC.

Part 1 of the Task 2 report presents the detailed design studies for the coastal protection options (Figure 1.1) and Part 2 (this document) describes the "Construction Methodology, Cost Estimate and Schedule". This report identifies some of the key aspects for the construction programme. It includes a description of the likely construction methods for each of the following elements:

1. Nearshore wedge reef
2. South reef
3. Beach nourishment using 450,000 m^3 of sand

The nearshore wedge reef and the south reef specifications are given in Table 1.1 and Table 1.2.

Figure 1.1 Puducherry beach restoration interventions - beach nourishment using 450,000 m^3 of sand, nearshore wedge reef in the north which acts as a sand retention structure as well as allowing sand bypassing to the north and an offshore reef at the south at 300 m north of the new pier groyne to increase the longevity of the beach nourishment.

Table 1.2 Southern reef specifications

Reef Information	
Reef orientation	10^{o} east of north
Reef length	200 m
Reef width at base	38 m
Reef height above seabed	5 m
Reef volume	12,000 m^3
Reef footprint	7,600 m^2
Reef crest elevation	0-1 m above CD
Depth at reef	4 m

2 Design Wave Conditions

The maximum deep water significant wave height off the Puducherry coast was 5.36 m. The hindcast wave heights are in good agreement with local measurements, the wave rider buoy wave measurements at 8 m and at 30 m water depth recorded more than 8 m during cyclone Thane. Thane was an exceptional event with large waves for a very short duration at Pondicherry. Otherwise, the hindcast and measurements were in good agreement.

In general, significant wave heights are typically 0.5 - 1 m and occasionally reaching 2-6 m during the northeast monsoon (Figure 2.1). The large wave heights are associated with cyclones and extreme storms, with wave directions from north of east.

A summary of monthly wave statistics is presented in Table 2.1. A very strong seasonal signal can be seen in the wave record. Larger waves occur during the beginning and end of the year and at the peak of the NE monsoon season (December - January).

An extreme value analysis on the 36-year offshore wave hindcast data set was computed. The location of the hindcast data extractions point is 12^{o}N and 80^{o}E, about 21 km northeast of the Puducherry Harbour This analysis provides an estimate of wave heights for different recurrence intervals (RI) based on the long-term data. The results of the analysis are listed in Table 2.2

Extreme wave height statistics reveal the exceptionally large events. Also of interest are the continuous periods of time during which waves do not exceed a specific wave height - known as periods of continuous non-exceedance. Analysing these periods for different months is necessary for the identification of the best period for construction and the likely duration of periods with small waves. The average and the maximum of all the periods of continuous nonexceedance were calculated for each month of the year.The duration of non-exceedance was calculated using the 36-year offshore hindcast data. The results are tabulated for critical wave heights of 0.5 m, 1 m and 2 m (Table 2.3).

The mean durations of continuous non-exceedance were reasonably short (maximum of 36 hours and minimum of 8.8 hours) meaning that the significant wave height only falls below the critical wave height threshold for short periods of time at Pondicherry

For the 1 m critical wave height, the maximum duration of continuous non-exceedance was 744 hours in March, July, August and October and the corresponding longest mean duration was for October with 253.6 hours. The shortest maximum duration of non-exceedance was for the month of December with 330 hours and the shortest mean duration of non-exceedance was for the month of May with 34.6 hours.

It appears from these results that the months of February to October are suitable for construction although the variation is not very large between months and the duration of periods below the critical wave heights is relatively short in all months. The months of January and December are the least suitable for construction.

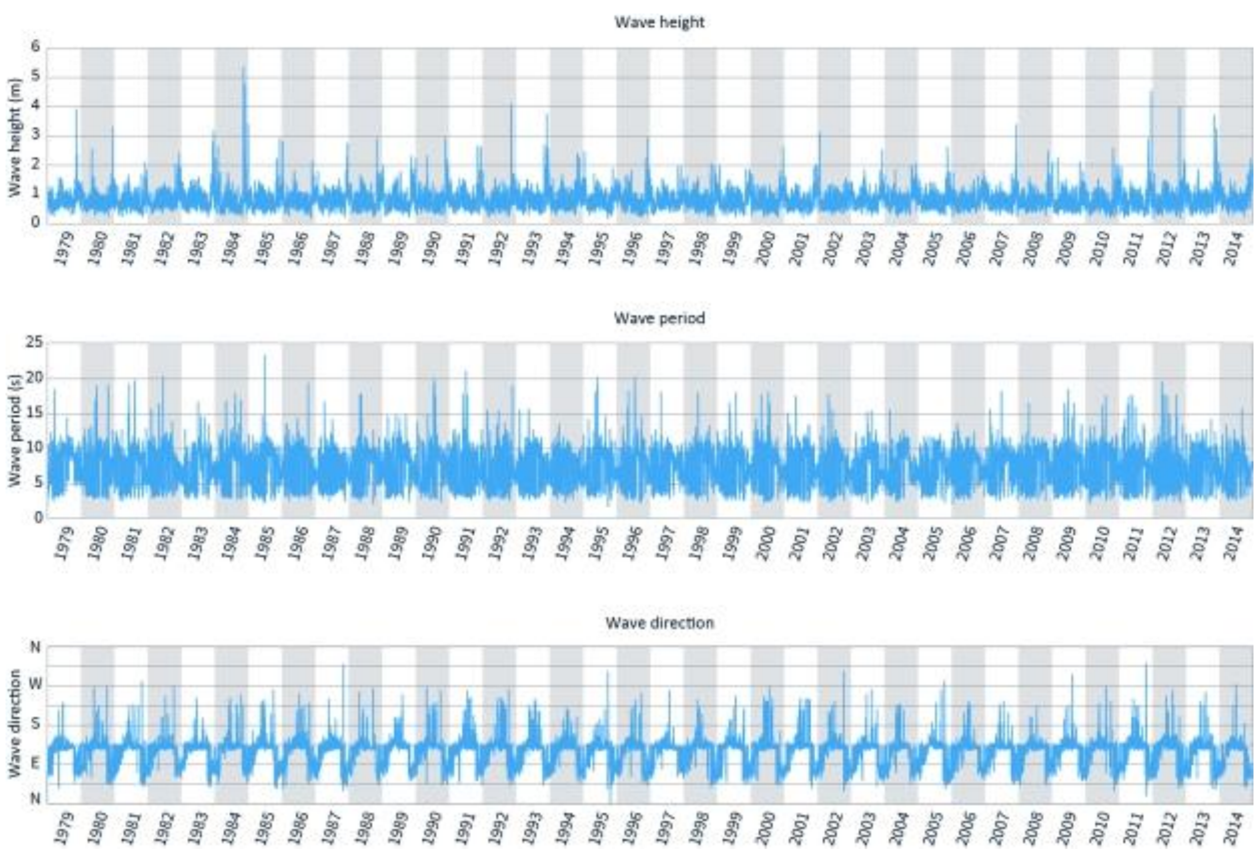


Figure 2.1 Time series of offshore wave height (top), wave period (middle) and direction (bottom) from January 1st, 1979 to December 31st, 2014 (36 years offshore hindcast data at 3 hourly time step). The hindcast data extractions point is, about 21 km northeast of the Puducherry Harbour

Table 2.1. Monthly averaged and monthly maximum for wave height and period from the 1979-2014 time series. Monthly averaged wave direction from the 1979-2014 offshore hindcast data. The hindcast data extractions point is, about 21 km northeast of the Puducherry Harbour

Month	Wave Height (m)		Wave Period (s)		Wave Direction (degree)
	Average	Max	Average	Max	Average
January	0.9	2.8	6.6	13.8	94
February	0.8	3.1	6.9	15.7	114
March	0.7	2.2	6.8	18.5	133
April	0.7	2.0	6.3	19.6	144
May	0.9	2.6	6.9	18.3	152
June	0.8	1.9	8.0	20.4	158
July	0.7	1.8	9.1	23.4	152
August	0.7	1.5	9.2	19.7	148
September	0.7	1.5	8.9	17.7	147
October	0.6	4.0	8.3	19.4	138
November	1.0	5.4	7.2	19.0	100
December	1.1	4.8	6.8	13.4	89

Table 2.2 Extreme event analysis for wave height computed from the 1979-2014 time series using a Weibull distribution function (shape coefficient k=0.75, R=0.995). The hindcast data extractions point is, about 21 km northeast of the Puducherry Harbour

Return period	1 y	5 y	10 y	25 y	50 y	100 y
Wave height	3.6 m	4.3 m	4.6 m	5.1 m	5.4 m	5.7 m

Table 2.3 Monthly mean and maximum duration of continuous non-exceedance of 0. 5 m, 1 m, and 2 m wave height from the 1979-2014 offshore hindcast data. The hindcast data extractions point is, about 21 km northeast of the Puducherry Harbour

Threshold	0.5 m		1 m		2 m	
Month	Mean duration (hours)	Maximum duration (hours)	Mean duration (hours)	Maximum duration (hours)	Mean duration (hours)	Maximum duration (hours)
January	23.9	144.0	95.9	576.0	620.2	744.0
February	18.6	87.0	188.1	696.0	658.0	696.0
March	18.5	156.0	204.6	744.0	723.6	744.0
April	11.8	165.0	88.0	720.0	720.0	720.0
May	9.0	69.0	34.6	423.0	704.1	744.0
June	8.8	66.0	53.3	456.0	720.0	720.0
July	9.2	90.0	117.3	744.0	744.0	744.0
August	10.5	108.0	117.1	744.0	744.0	744.0
September	17.5	123.0	143.3	720.0	720.0	720.0
October	33.2	306.0	253.6	744.0	703.0	744.0
November	36.0	207.0	101.2	657.0	375.3	720.0
December	32.9	105.0	60.3	330.0	340.6	744.0

2.1 Tides

Time series plots of water level from various short-term (15-30 day) nearshore deployments show the tidal range at Puducherry is 1.07 m. The mean sea level is 0.55 m

3 NEARSHORE WEDGE REEF

3.1 Introduction

The designed nearshore wedge reef layout is shown in Figures 3.1 and 3.2. The reef is a triangular shape caisson resting on a bed of rocks laid horizontally at 2.5 m depth. The caisson width is 60 m and the spine at Chart Datum is 60 m long. The offshore tip of the rock bed is at 4.5 m level. The layout plan is depicted in Figure 3.3 with the cross sections drawn in Figure 3.4.

The Wedge section will be a maximum of 2.5 m above the rock bed. The reef crest is designed to be at the water surface at low tide and submerged by more than 1 m during high tide. The reef location is indicated in Figure 1.1. To bring the reef seaward and to create an essential construction zone, a work area is recommended at the shore which is approximately 85 m long (longshore) and 32 m wide (cross-shore) (Figures 3.1 and 3.2). The work area would be walled and filled with sand. The work area has a wall height of 3 m above datum on the ocean side and tapers up to boardwalk level on the landward side for public convenience and easy access.

Figure 3.1 Detailed design of the wedge reef. The reef is a single concrete unit resting on a bed of rocks laid horizontally at -2.5 m. The work area is shown in the lee and the gap between the reef and work area is filled with rocks to Chart Datum level.

Table 3.1 Nearshore Wedge Reef specifications

Reef Information		
Reef orientation	17^{o} south of east	
Reef volume	10,500 m^3	
Reef footprint	6,300 m^2	
Max reef elev. above seabed	4 m	
Reef crest elevation	Chart Datum	
Depth at offshore tip	4.5 m	
Length of spine	80 m	
Max width of reef	82 m	
Cross-shore length	120 m	
Work area length and width	L=85 m	W=32 m
Area of work area	2970 m2	
Sand volume for work area fill	Approx. 9,000 m^3	

3.2 Zones to be considered

In Figure 3.2, the zones to be constructed are named for ease of description in the text:

- Zone 1 is the sand-filled beach in the work area
- Zone 2 is the rock retaining wall around the work area
- Zone 3 is the sloping ledge at the south in the lee of the wedge with elevations from Chart Datum to -2.5 m
- Zone 4 is the sloping ledge at the north in the lee of the wedge with elevations from Chart Datum to -2.5 m
- Zone 5 is the ledge at Chart Datum between the reef and the work area
- Zone 6 has been identified by the modelling as being subject to higher waves on the northern side of Zone 4 (Figure 3.6)
- Zone 7 is the toe scour apron at -2.5 m around the wedge

Zone 8 is the sloping berm from the scour apron

- Zone 9 is the wedge itself.

Cross-section 3 through the gap between the work area and the wedge consists of an armour layer on 1:2.5 slope on both the north and south sides and toe berms of 3 m wide (Figure 3.6).

3.3 Rock sizes

In summary, all of the rock areas have two layers:

- the 0.5 m thick filter layer at the seabed consisting of 5-10 kg stone.
- the armour layer which consists of 1.52 t rocks.

The design wave heights for each zone are presented in Table 3.2. The depths allow 1.1 m above CD for the high tide and 0.4 m for storm surge and set-up for a total of 1.5 m. Many of the zones are steep around the reef and so we have assumed a height to depth breaking criterion of 1.0, i.e. the breaking wave height is equal to the water depth. Each zone is described as exposed or submerged, which determines if the Hudson or the Van der Meer formula are used for rock stability calculations.

Table 3.2 presents design wave heights for each of the zones and considers their likely exposure. Most segments are rarely exposed, only at low tide under the trough of a wave reaching the zone.

The rock stability formulae acknowledge that the sizes are much less when the rocks are underwater. Most damage is done when plunging waves breaks directly onto exposed rocks. Thus, while the design wave heights might be bigger in some locations, the wave force is greatly reduced because the zone is more submerged.

Maximum wave height over the entire structure can be expected during high water level conditions (such as high tide coinciding with storm surge). However, the structure's submergence also increases under these conditions. While applying the formula for armour stone size calculations, the low water level tends to increase the size of the stones, but the high water level or increased submergence of the structure tends to reduce the rock size. For the design, the armour stones sizes have been computed for both high water level with high submergence and for low water levels, when the submergence is marginal.

After taking account of these factors using the design formulae given in Appendix 1, it was found that the 1.52 t stone would be adequate for all of the reef areas and 0.5 t stone for the work area wall. At depth on the toe scour berm (Zone 8), a smaller stone of 163 kg is recommended.

We note the following

- While a 3 m wide toe berm has been added already, Zone 6 might ultimately prove to need larger stone or a wider berm. If so, we recommend the addition of concrete tetrapods at a later stage.
- Berm 4b has been extended on the north side of the Work Area back to the beach because this area is shown by the model to be sometimes (rarely) scoured and it's subject to direct wave attack during the NE monsoon. Some monitoring of this area will reveal if further reinforcement is needed. The south side is predicted by be completely buried by the nourishment and so no additional stone berm has been added there.
- Several areas will need excavation before placement of the filter layer. These are shown in the cross-sections (Figure 3.6).

Table 3.2 The design wave heights and adjacent depths in each zone. The depth includes an allowance of 1.1 m above CD for high tide, plus a set-up and storm surge allowance of 0.4 m for a total of 1.5 m above CD.

Zone	Description	Depth (m)	Wave height (m)	Classification	Comments
1	Work area beach	Above water level	0	Emerged	Sandy beach
2	Work area rock wall	1.5	1.5	Emerged	Partially protected by reef. Exposed on north side. South side may be filled with sand.
3	South slope	3.5	3.5	Emerged at low tide	May be buried by sand. Limited direct wave attack
4a	North slope	4	4	Emerged at low tide	Protected by the scour apron
4b	North slope	3.5	3.5	Emerged at low tide	Potential for erosion of sand bed
5	b/n reef and work area	1.5	1.5	Submerged mostly	Protected by the reef except for the northern zone
6	High waves	4.5	4.5	Emerged at low tide	Shown by model to receive higher waves under NE monsoon in particular
7	Toe scour apron	5.0	5.0	Submerged	Maximum wave heights under 30-50 year extreme conditions
8	Toe Berm	5.0	5.0	Submerged	Maximum wave heights under 30-50 year extreme conditions
9	Wedge	4.0	4.0	Emerged crest at low tide	Protected by scour apron

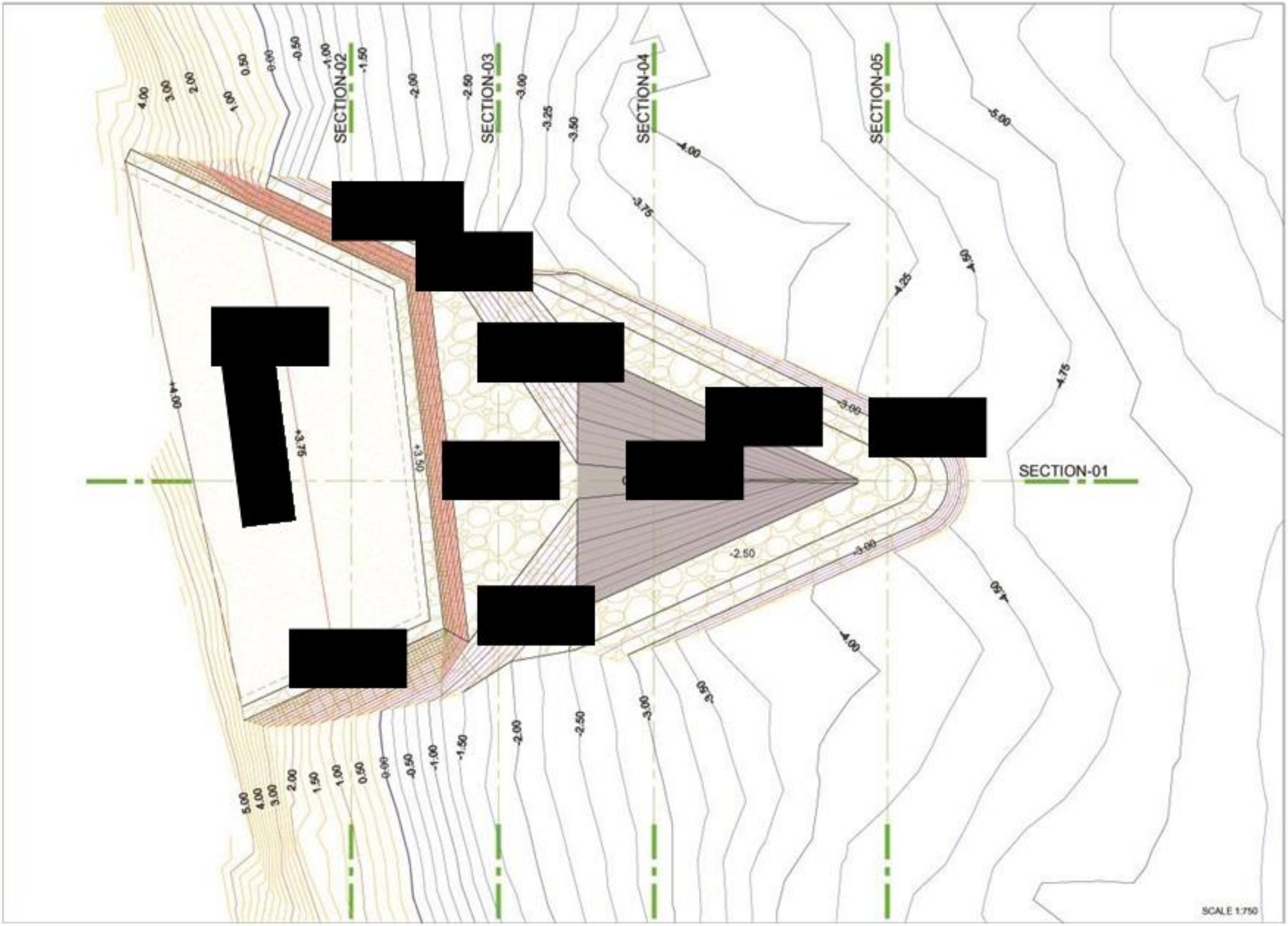


Figure 3.2. Wedge reef layout plan and bathymetric contours. Depths are in metres and reduced to Chart Datum (Survey March 2015)

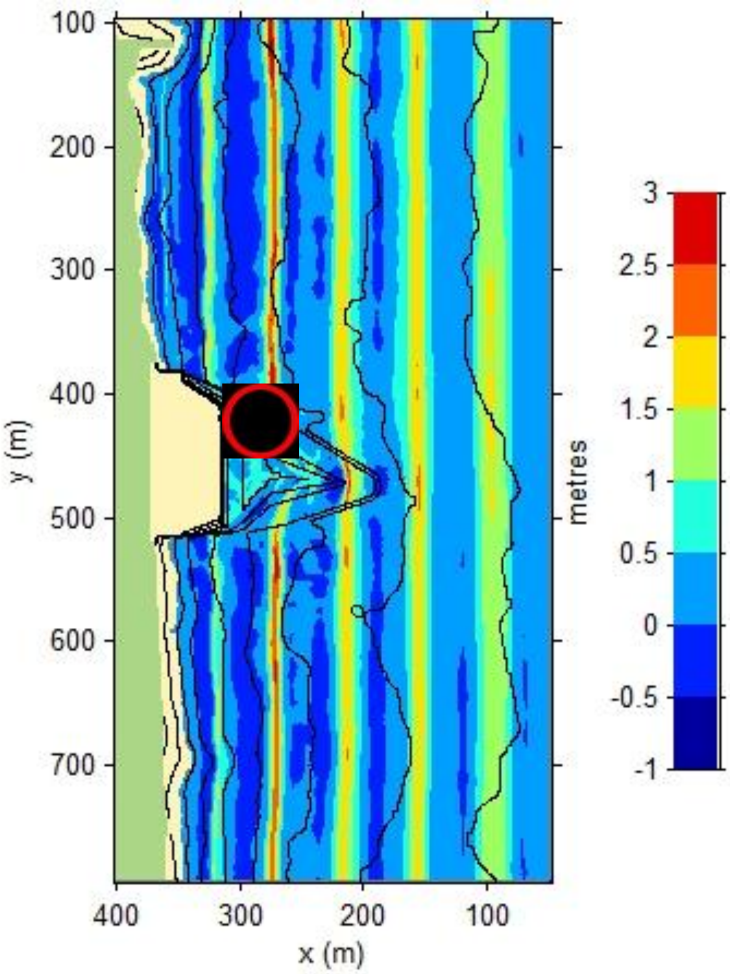


Figure 3.3 Overview of wave patterns around the reef. Red circle shows the critical area of the rock base

3.4 Design of the work area wall

The Work Area wall consists of a bed of filter stone (5-10 kg in a layer 0.5 m thick) with the armour units on top. Geotextile will be placed in the lee of the wall and the whole area will be filled with sand.

The wall will be subject to direct wave attack during high tide. On the basis of Hudson formula, the seaward gradient of the wall will be 1V:2.5H. Weight of the armour unit estimated from the Hudson formula is 0.5 t and number of units per cubic metre is 3.4.

The crest of the work area wall is at + 3 m above CD. The filter layer needs excavation to -2.5 m, where the natural depth is around -1.5 m (Cross-section in Figure 3.6).

3.5 ROCK PLATFORMS

Structural slope stability is critically dependent on toe support. To give adequate protection to the caisson, a wide scour toe at -2.5 m below CD has been designed. The scour toe is 23 m wide in front of the caisson. Scour is predicted by the modelling and so we expect some of this rock to subside. Thus, we have added a toe berm of smaller rocks to reduce the scour and protect the toe. The slope of the berm is 1V:2.5H. . The toe berm is at -4.5 m on the front of the reef and grades up to shallower depths shorewards. The median weight of the armour stones on the toe is 1.52 t and 163 kg on the berm.

As noted above, the other zones also require the filter layer and the armour of 1.52 t.

Further details about the foundations can be seen in the cross-section drawings (Figures 3.5-3.6).

3.5.1 Performance of tightly packed armour layers

Stewart (2002) demonstrated that the stability of armour layers increases significantly if rock armour is placed closely to achieve a tight packing. Moreover, dissipation of wave energy was not greatly affected by this reduced porosity (within the range tested). Accordingly, tightly packed armour layers are recommended in the present design

For tight packing, each armour stone will need to be placed and so the construction depends on the number of stones but is independent of the mass. While the purchase and transport cost of larger stones is higher, the work time for placement is reduced.

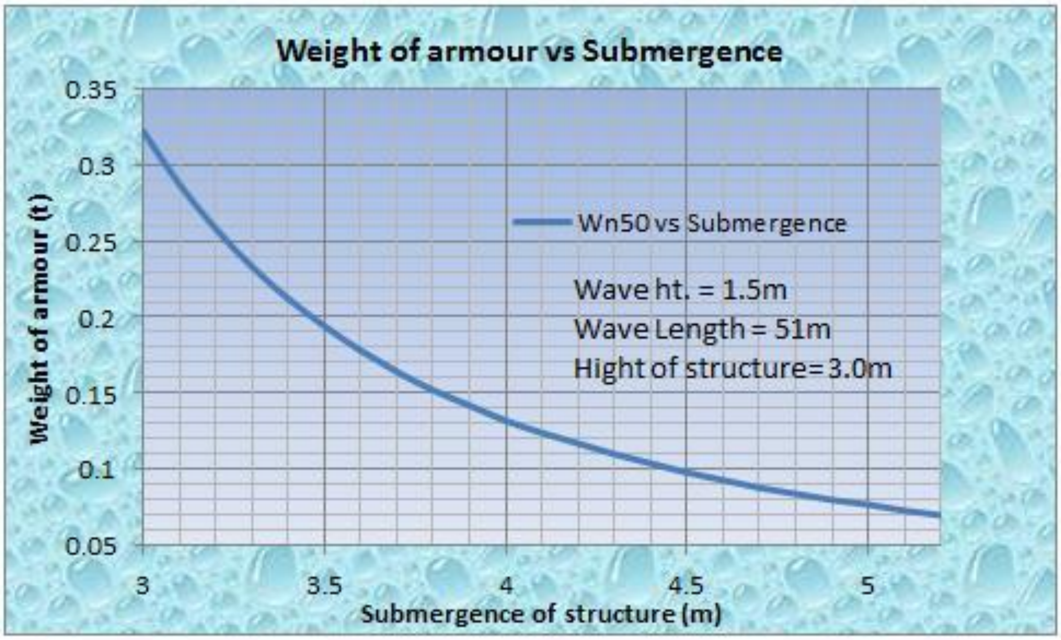


Fig 3.4. Plot showing variation of stone weight vs submergence

Table 3.3 Rock size requirements

	Rock size		
	Armour layer	**Toe Berm**	**Filter layer**
Work area wall	0.5 t	0.5 t	5-10 kg
Other zones	1.52t	163 kg	5-10 kg

3.5.2 Rock grading

When using wider than normal gradings (D_{85}/D_{15} < 1.5), there is greater potential for the smallest rocks to become dislodged from the body of the structure, which will ultimately lead to a decrease in the stability of the armour layer. The grading of the stones to be used in the wedge reef's construction should be as follows:

1.52 t stones : 75 % stones should be higher than 1.5 t

500 kg stones : 75% stones should be higher than 450 kg

163 kg stones : 75% stones should be higher than 150 kg

5-10 kg stones : 75% stones should be higher than 6 kg

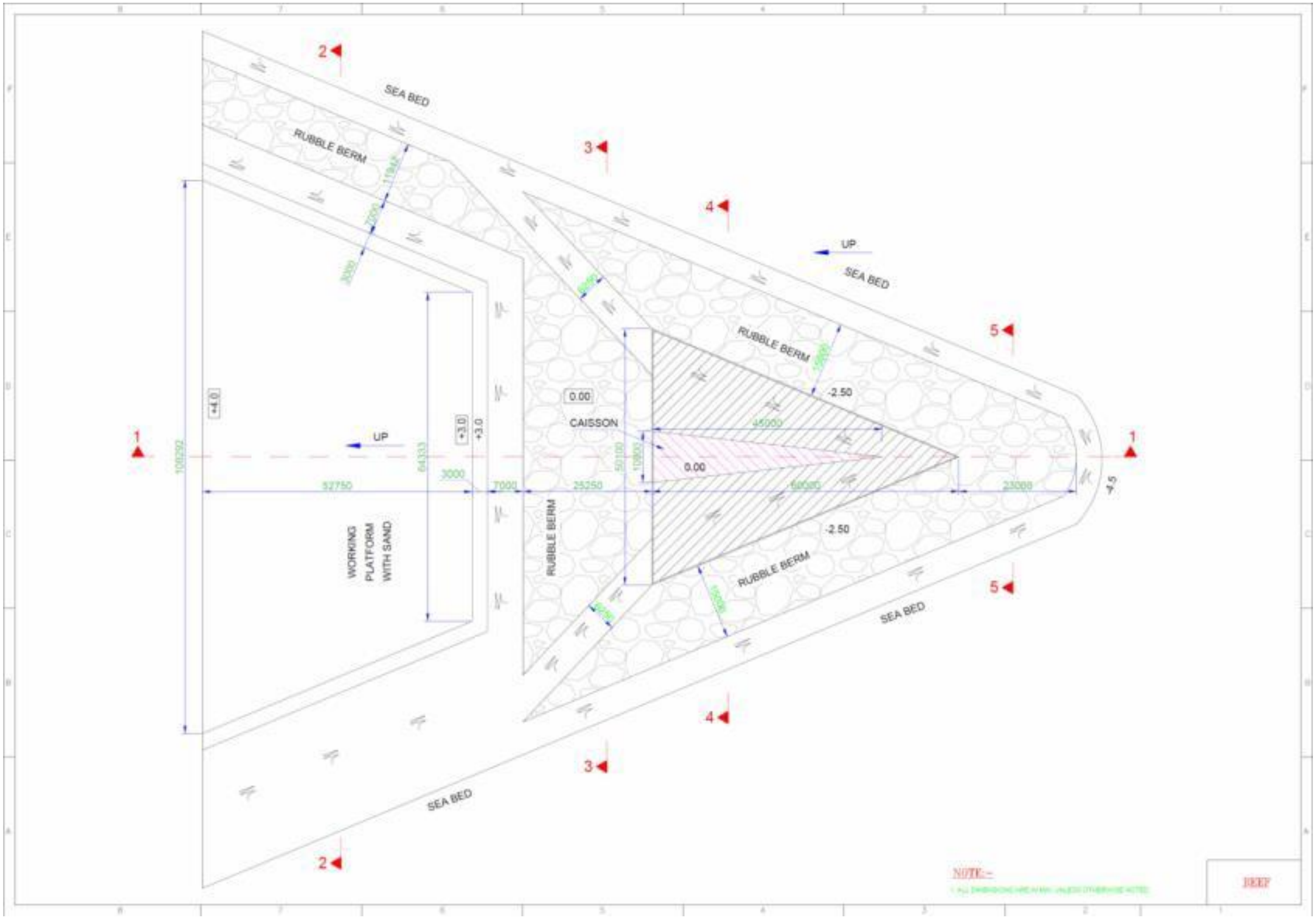


Figure 3.5 Layout plan of the nearshore wedge reef

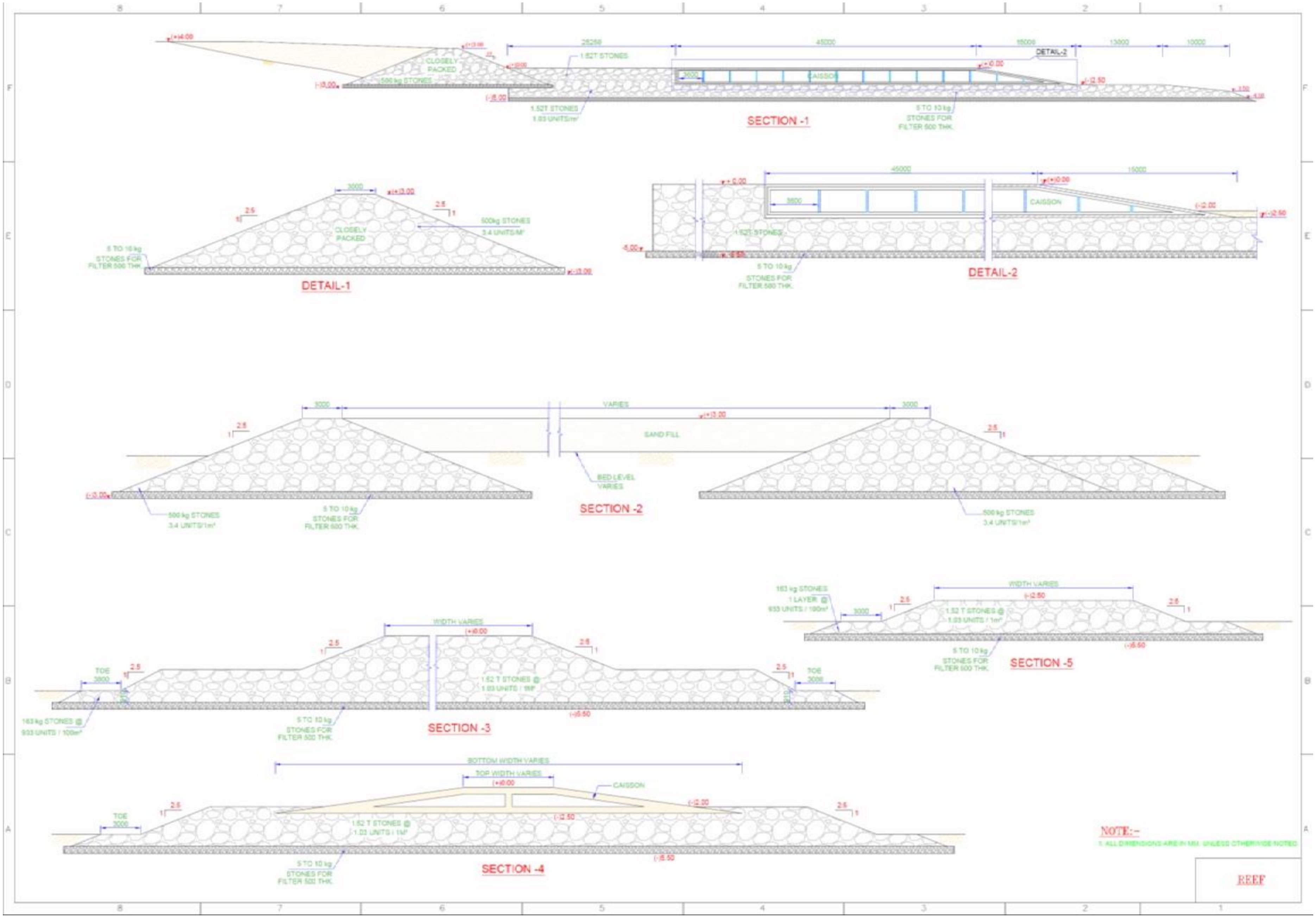


Figure 3.6 Reef cross-sections

3.6 The Wedge

The Wedge component is a triangular shape laid on top of the horizontal rock platform. The materials considered for the Wedge construction are HDPE sheets, fibreglass, concrete and steel sheets. Though HDPE sheets are light and can withstand the wave forces, non-availability of wide sheets and the requirements for a large number of joints and welding made it unfeasible for the Wedge construction. Fibreglass was also too expensive. The materials considered are concrete and steel.

3.6.1 Concrete Wedge

The critical factor for the design of the concrete caisson is that it should be structurally stable while being practical for construction. This includes floating to the site during the construction stage, ability of the structure to withstand hydrostatic pressure, costs and minimum maintenance requirements. The external wall of the concrete caisson should have sufficient thickness to withstand hydrostatic pressures, but thin enough to float the structure. So, the structural designer has limited freedom to provide a heavy section for the sake of structural safety. In order to float the structure as a single unit, the elements of the caisson should be light enough so that the weight of the water displaced is less than the weight of the structure. This has been achieved through a process of iteration.

At the initial trials with thick walls for the concrete Wedge, the buoyancy was less than the weight of the caisson. So the thickness of the top and bottom slabs has been optimised by reducing the internal beam and column spacing. At three locations longitudinally and transversely, there should be stiffeners for the structural integrity. Providing a vertical slab would be easy but would add to the weight and jeopardise its buoyancy. Further, diagonal columns are required to replace the heavy slab. To float the caisson as a single unit, the maximum thickness for the bottom and top slab is 150 mm. The plan and cross-section of the Wedge designed as a single unit are given in Figure 3.8 and Figure 3.9.

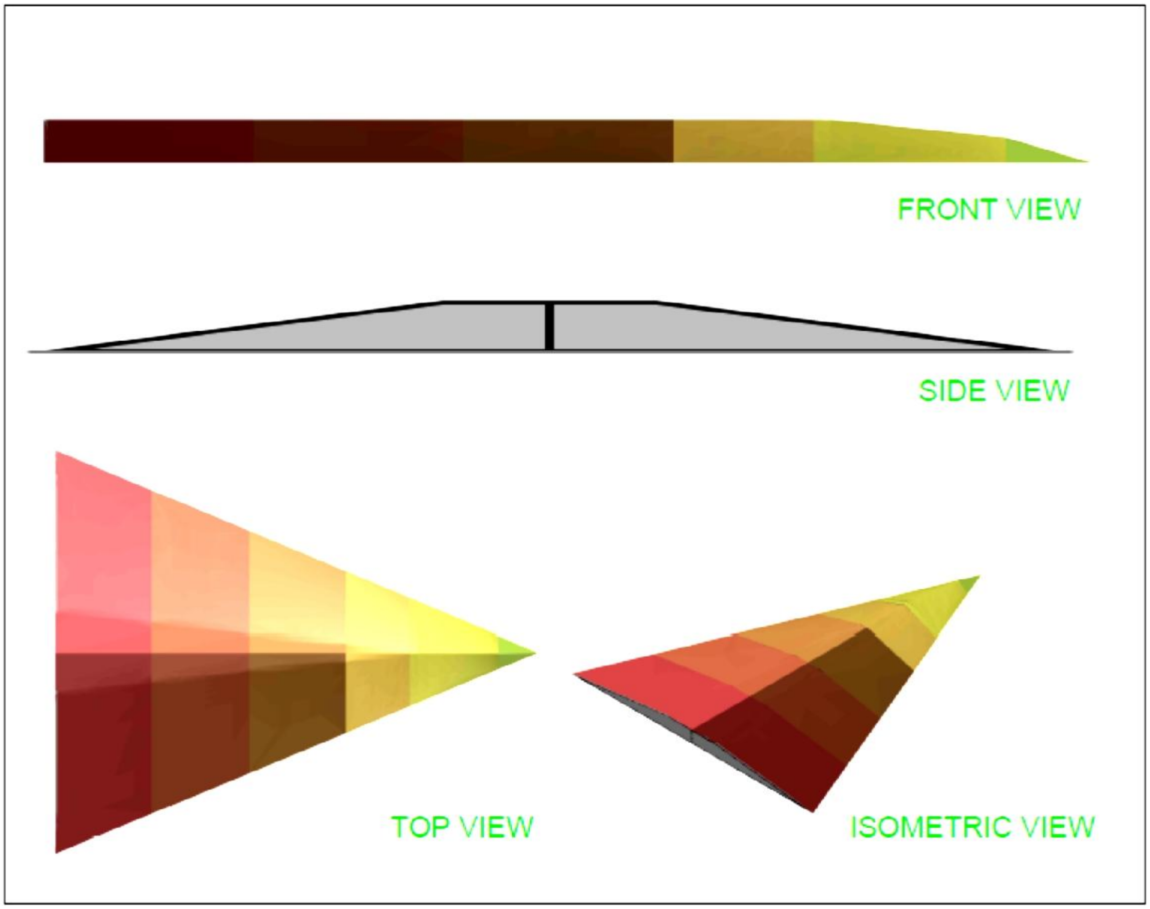


Figure 3.7. 3D views of the caisson

If the wall thickness of the caisson is increased, there is a gross shortfall in the buoyancy and the structure cannot float as a single unit. Further, even with 150 mm thick walls, the centre of gravity and centre of buoyancy have an offset and it is likely that the caisson will tilt backwards slightly while floating. This has to be corrected with sand filling or other methods in the front during trials. It is to be noted that the 150 mm wall thick caisson will be floating but submerged almost to the crest level. In addition, a large dry dock is required to cast the caisson and to test the buoyancy requirements before floating the structure to the site for placement. The cost of the dock for casting the caisson as a single unit is very high (about INR 16 crores) and hence it's not an economically viable option. Due to the shortfall in buoyancy and the triangular shape of the Wedge, floating the concrete structure for placement is ruled out.

We considered the option of constructing the concrete Wedge as a single unit at the work area and sliding it into position. However, this will not be practical given its weight (about 2500 t). The construction difficulties, particularly floating the heavy caisson and its accurate placement at the reef site make constructing the Wedge as a single unit from concrete unfeasible.

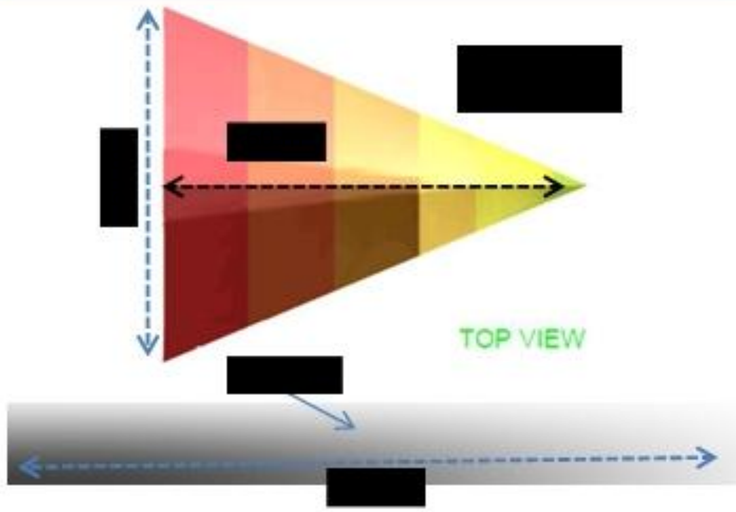
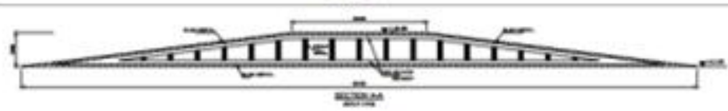
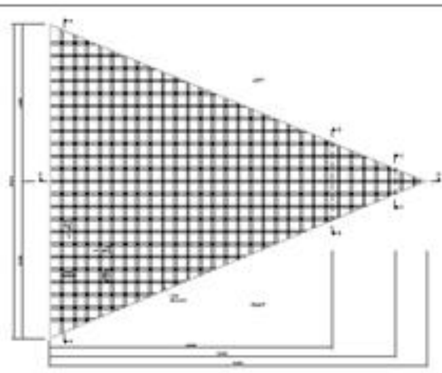
An alternate option is to cast the Wedge in multiple units and placing the sections individually. Accordingly, the Wedge design is modified by dividing the caisson into 8 individual blocks. The slab thickness is 200 mm and the weight of the individual blocks varies from 121 T to 326 T. The plan and cross-section of the modified Wedge sections is given in Figure 3.10.

The casting of the blocks can be done on land.

The installing of an overhead crane would rest in piles hammered into the seabed on both sides of the wedge. We haven't added this to the budget. At least, large cranes and excavators for lifting and placement of the units will be required. In our view, this option may ultimately prove to be too expensive and the final result will not be satisfactory. The final cost is unlikely to be precisely known until a contractor is hired. Also, accurate placement of the caisson units is very important to eliminate gaps to prevent water movement and to protect the rock bed layer below from scour by currents and wave action. This may prove to be difficult over the irregular rock base. However, the weight of the caissons may ultimately cause the rocks to level out and the problem may not be terminal. Inter-locking fittings (male/female) in the adjacent caissons would be needed.

In all cases, the concrete caissons would need to be filled with sand to be stable.

Table 3.4 Cross section and main physical parameters for the 150 mm thick caisson concrete elements

Caisson	Physical parameter	
	Concrete Caisson Weight	1853 T
	Caisson Submerged Buoyancy	2084 T
	Net buoyancy	230 T
	Top slab thickness	150 mm
	Bottom slab thickness	150 mm
	There are no inner walls	
	Edge wall thickness	150 mm
Note: Centre of gravity of the caisson is at 20.2m from the rear. Centre of buoyancy is at 25.7m. This will induce a residual moment. The caisson will slightly tilt, the rear going down. This can be rectified by sand filling in the front or buoyancy tanks.		

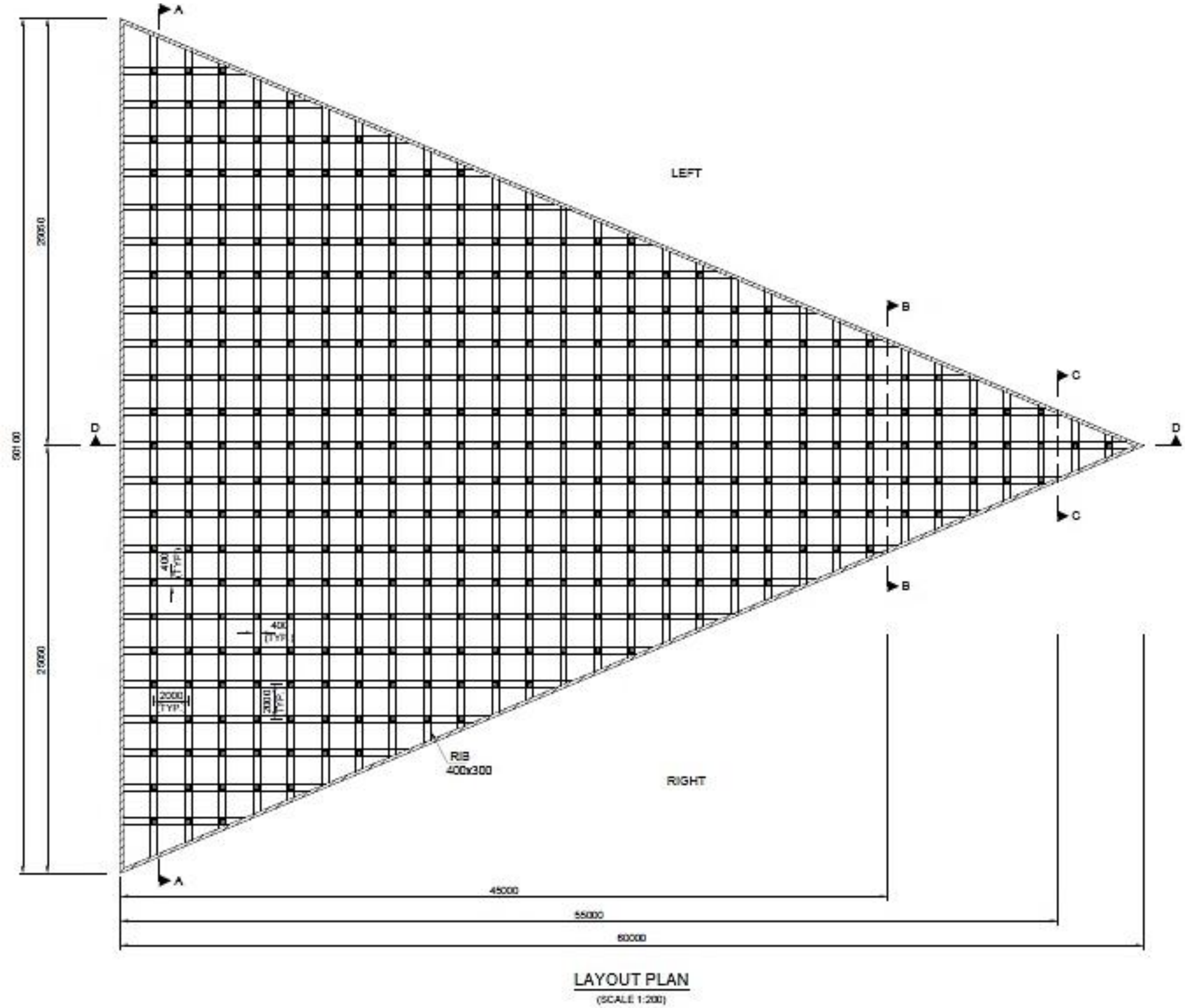


Figure 3.8. Layout plan of concrete caisson of 150 mm wall thickness and internal beam spacing of 2 m

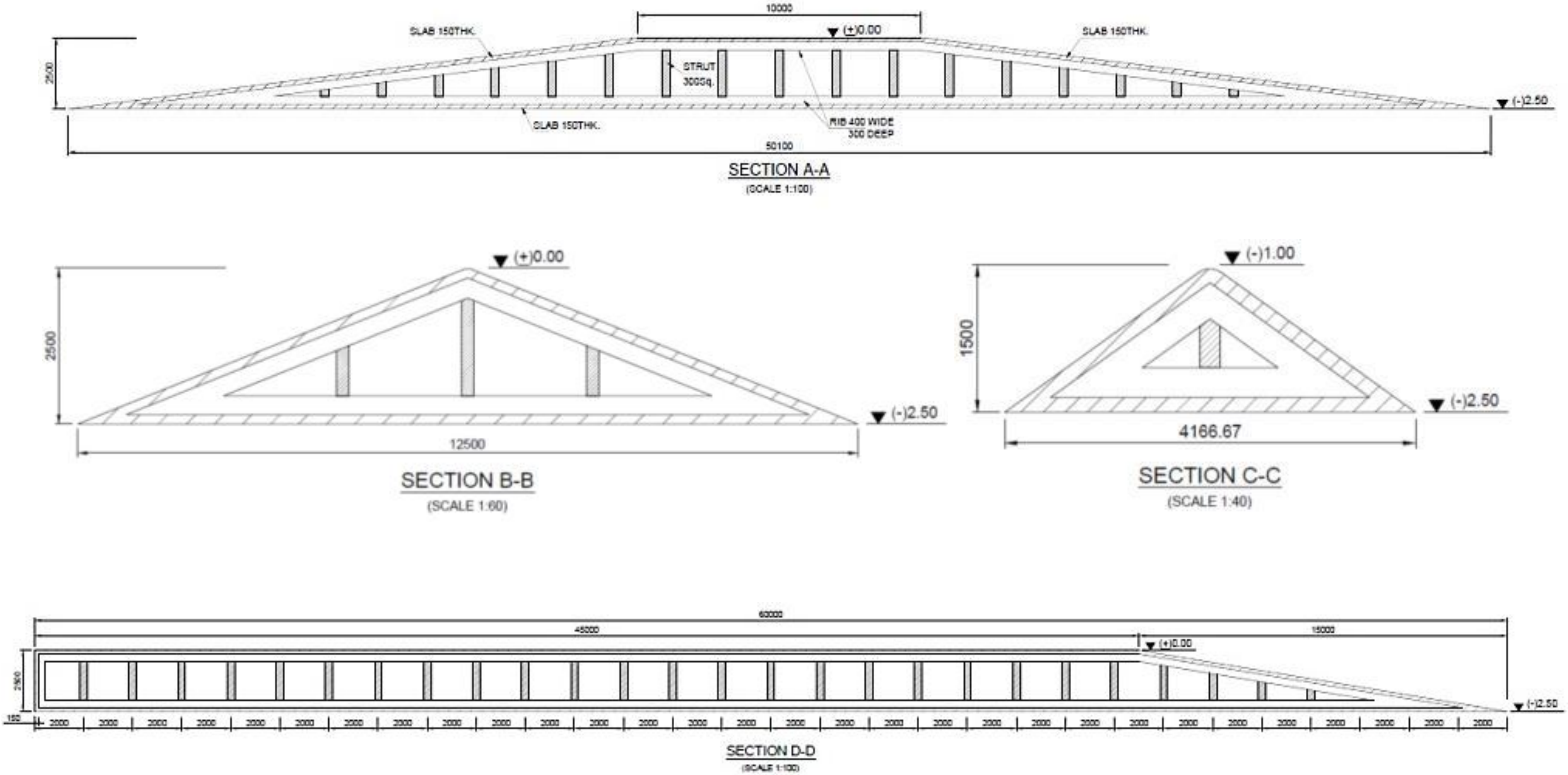


Fig 3.9 Concrete caisson (150 mm wall thickness) cross-sections (refer layout plan in Figure 3.8 for the cross-section locations)

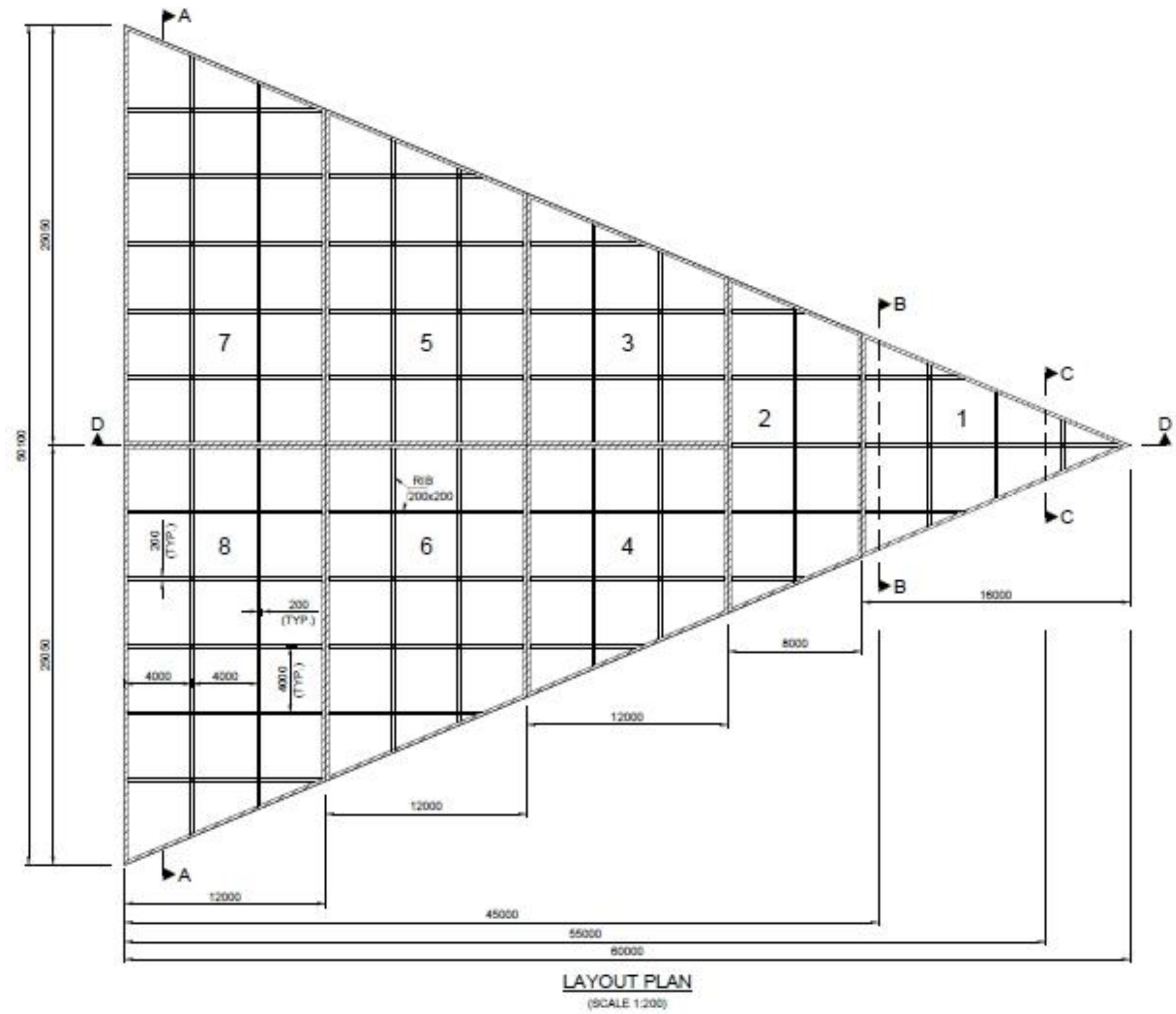
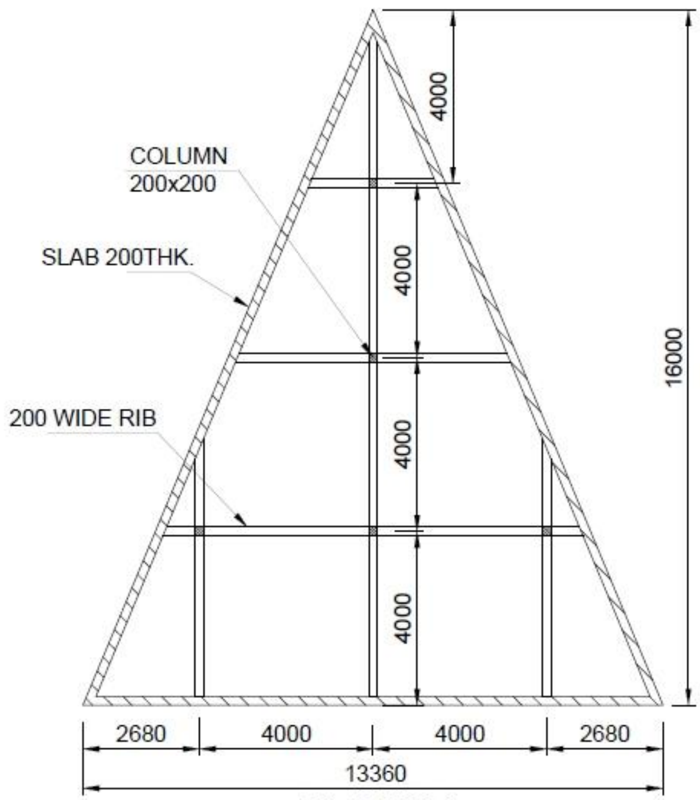
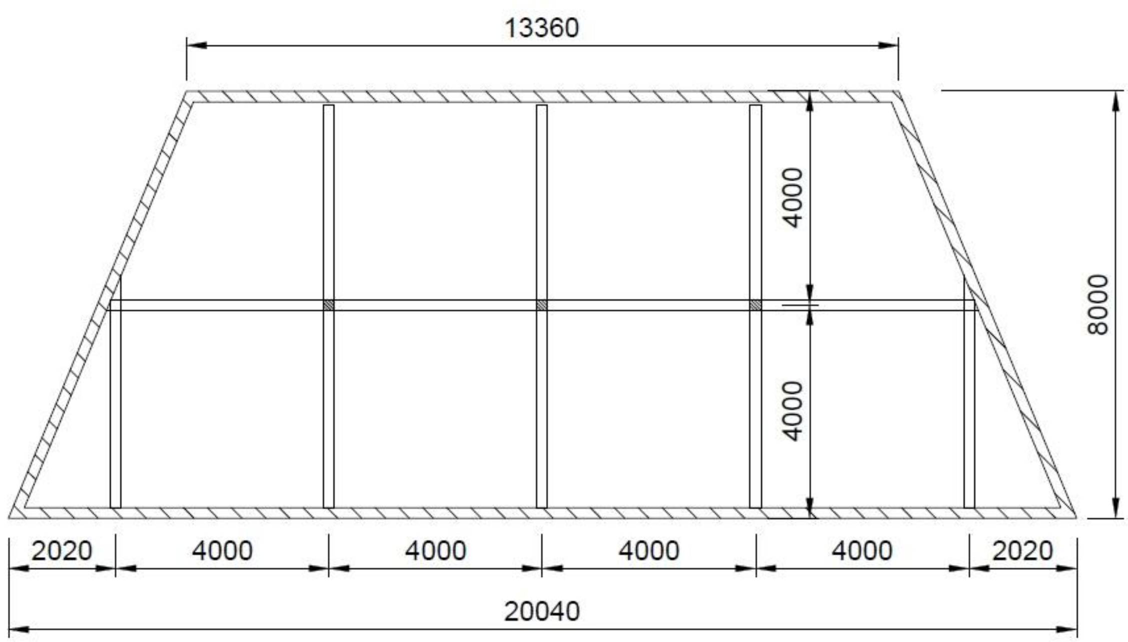


Figure 3.10. Plan view of concrete caisson with 200 m wall thickness. The caisson is to be constructed in multiple units.



BLOCK-1



BLOCK-2

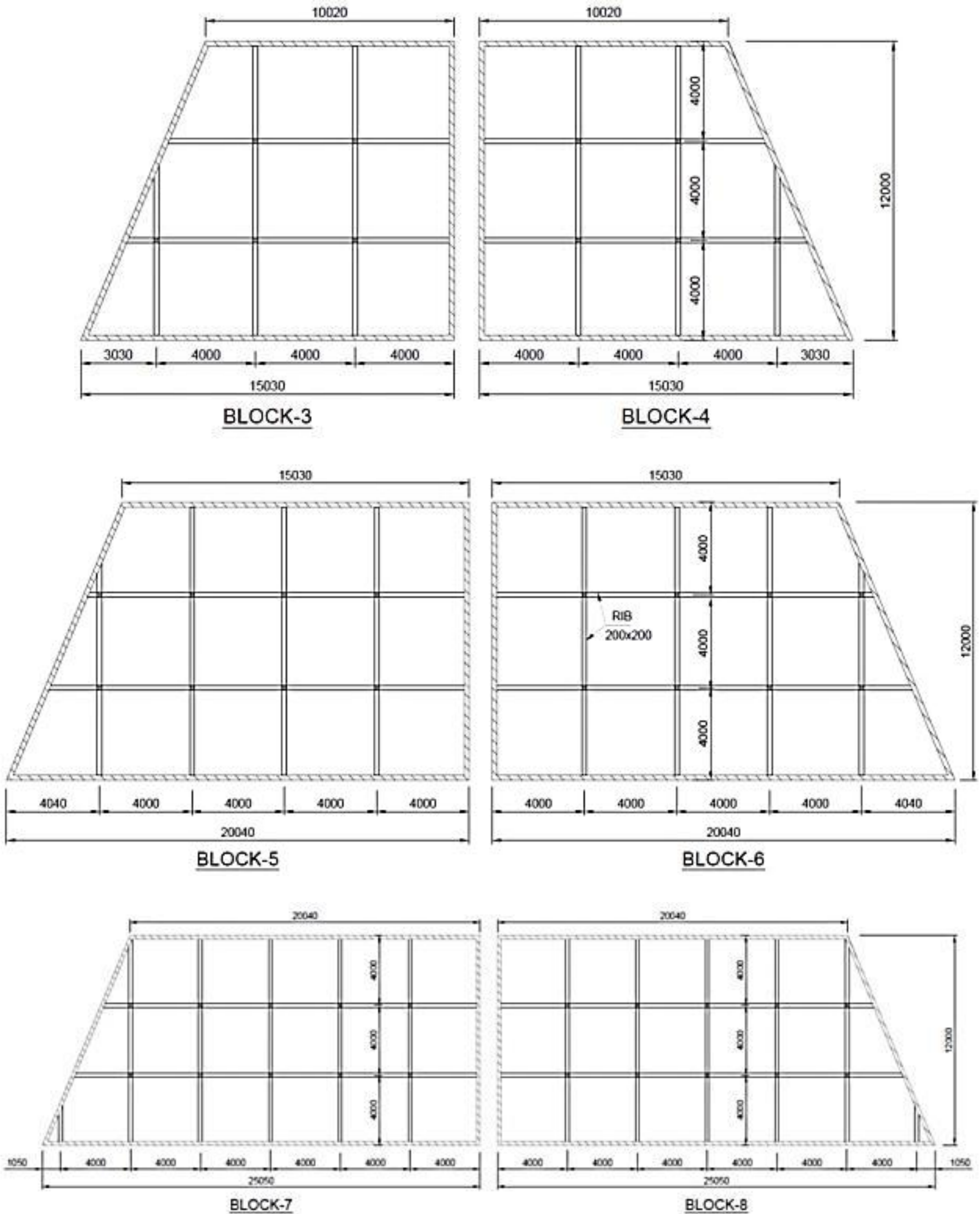


Figure 3.11 Concrete caisson blocks. Refer Figure 3.10 for the block sections. Note that the slab thickness is 200 mm

3.6.2 Steel Wedge

An alternative material considered for the caisson construction is 25 mm thick IS2062 grade steel sheets. The advantages of using steel sheets are that the entire caisson can be constructed as a single unit and floated to the site for placement. The steel caisson fabrication activities can be on land. The draught requirement for floating is less than 1 m and there's no need for heavy equipment for placement of the Wedge. The steel Wedge is stable if filled with water and can be re-floated easily for maintenance works or removal of the structure if required.

Corrosion of steel in the marine environment has been examined. Although slow corrosion of the steel plates will occur, there are measures for increasing the effective life of a steel structure by using (i) heavier section, (ii) sacrificial anodic protection and (iii) applying a protective organic coating, which can be used separately or in combination. All caisson welded connections shall be continuous to develop maximum strength and to facilitate cleaning and coating for corrosion protection Welding will be done on the inside to minimise exposure to oxygenated seawater. The plan and cross-section of the steel Wedge is given in Figures 3.12 and 3.13

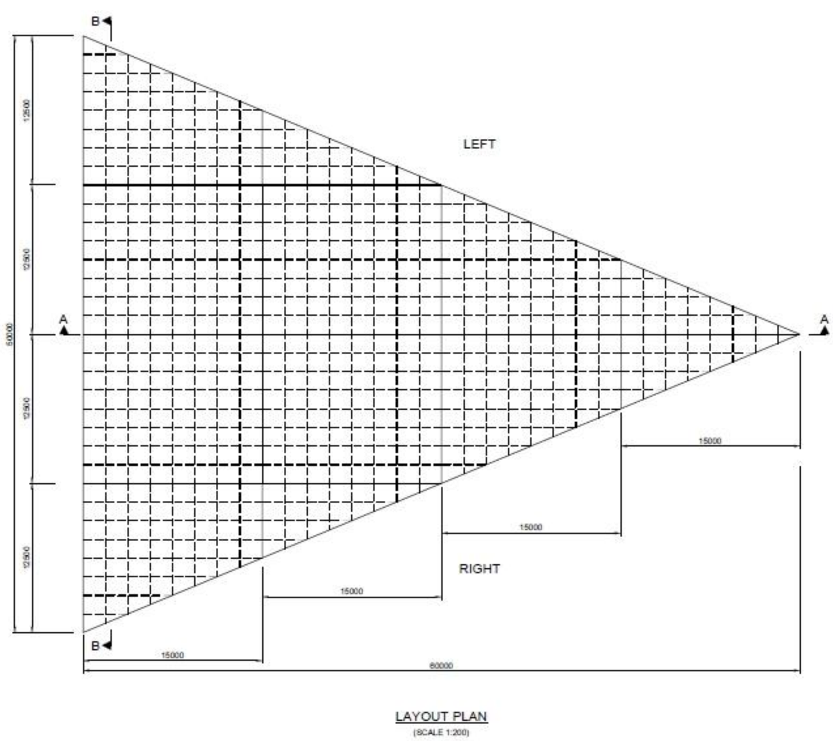


Figure 3.12. Layout plan of the steel caisson

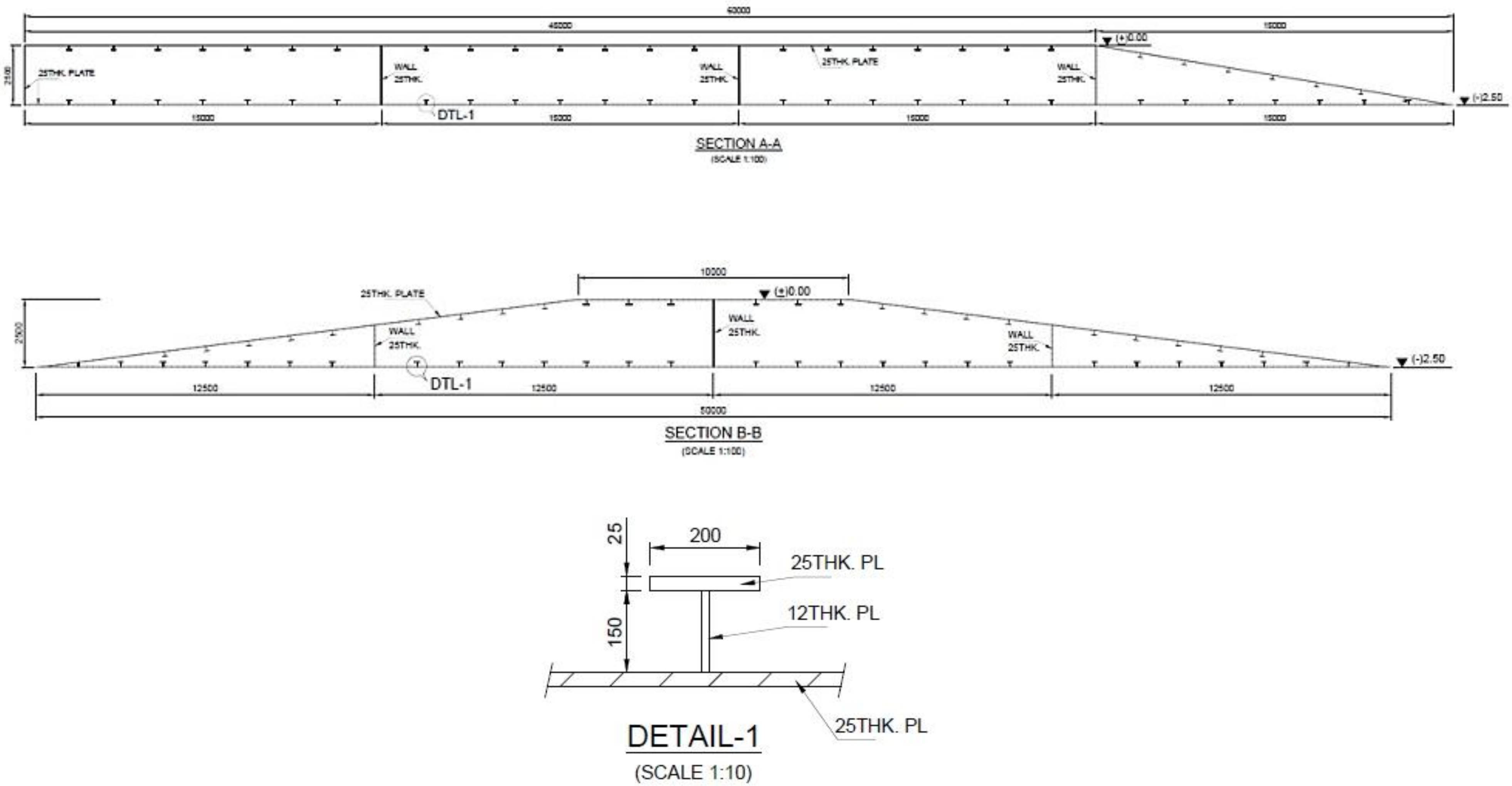


Figure 3.13. Steel caisson cross-sections

3.7 Construction Methods

Careful design and detailing of the rock platform, together with the reduction and appropriate allocation of risks are considered here to reduce construction time and costs. The use of simpler cross sections, with fewer rock gradings is used to reduce the number of construction operations and the degree of checking required. This in turn will make construction quicker and the use of a single grading of armour will also minimise the risk of damage to the structure during construction.

Construction duration often has a significant impact on construction costs and, and hence, opportunities for enabling maximum utilisation of plant and through night working primarily for trucking rocks to the site are envisaged. A good understanding of the working methods likely to be adopted and the influence of different issues on construction must be considered.

Prior to the construction the contractor should have the correct type of equipment and construction method and the structure needs to be evaluated with respect to:

- Layout plans
- Volumes and types of stones required
- Sand source and site delivery method
- Temporary stone storage facilities at the sites
- Quarry location and production
- Transportation from the quarry site
- Accessibility of the works for both land-based and sea based activities
- Casting of the steel caisson
- Corrosion prevention methods
- External conditions affecting the works - existing rock sea wall, city roads and promenade use, water depth, wave and wind conditions, monsoon and cyclone, etc
- Stability of the structure in its partially completed state
- Floating and placement of the caisson

Because specific conditions apply for every structure, the construction methods need to be tailored to the project. Also, the methods vary from contractor to contractor depending upon the type of equipment and plant they own and their previous experience. The following section gives a description of the likely construction methods, but the successful bidder is expected to give a detailed construction methodology based on his past experience in similar projects.

3.7.1 Working in the surf zone

Working in the surf zone can be particularly hazardous. The issues to consider include:

- Breaking waves which can capsize small vessels especially if power is lost. Consequently beach launching of contractor's vessels should be discouraged as the vessel could turn broadside and be at its most vulnerable. For larger vessels, using anchors to hold position can mitigate this risk. Field measurements show strong tidal currents, which can cause additional problems.

- A manned safety boat in the surf zone is a major hazard and its use should be carefully planned.

- There is limited tug access (i.e. restricted to shallow draught tug which means limited power to act in emergencies). Operations often rely on manoeuvring with ropes using winches, two sea moorings and two land moorings.

- If barges are used for sea-based works, the under keel clearance should be sufficient as large swells can be expected. There may be additional obstructions resulting from operations (i.e. rock placement). Correct vessel selection is important when working in the shallow wave breaking area.

- Access into the work site through road (e.g. to unload rocks) may difficult during day time

- Weather reports from established sources are very important for the surf zone. Local knowledge is useful and wave forecasts can be obtained from specialist sources.

3.7.2 Pre-construction surveys

Pre-construction surveys of the project site and its surrounding areas are required to ensure that there is no significant variation in the sea bed levels during the design and construction time. Also, because of the direct relationship between the survey techniques and payments, the client and the contractor should ensure that an accurate and fair approach to surveying is adopted that will lead to the requirements of the works, tolerance levels and correct method of payment for the work done.

Control points located in a safe position on stable ground close to the work site need to be established and checked regularly. Intermediate control points are needed close to the work area and these should be examined regularly for damage throughout the project implementation. Care is required to ensure that no confusion occurs between the local reference level and Chart Datum noting that all the drawings are relative to Chart Datum.

The contractor is required to undertake close grid bathymetric and topographic surveys prior to the construction to build the structures to the defined crest level and slope. If there is no significant (less than 0.2 m) variation between the design seabed levels and the pre-construction survey, the alignment of the structure needs to be established with the help of Total Station or Kinematic GPS. If there is more than 0.2 m variation between the design levels and the pre-construction surveys, the designers need to be consulted prior to any construction activity to check the functional performance of the structure.

On completion of the surveys and its approval by the engineer including the contractor's construction methodology, the construction can be commenced. The suggested construction sequence is:

1. Work area
2. Rock base
3. Caisson reef

3.7.3 Work area

The work area (85 m long (alongshore) and 32 m (cross-shore) and attached to the coast) shall be the first segment developed. The work area walls need to be constructed as per the design provided and filled with sand. For the work area construction, it may be preferable to use land based construction methods. Typical wall construction sequence includes dumping of stones by dump trucks, placement of armour layer by excavator and sand filling of the work area. Due to the exposure of the site to waves, it's suggested that the wall be constructed during the calmestsea conditions. The work area construction should also consider the requirements of access to the rock base and the access area width and slope should be sufficient for practical execution of the rock base and caisson works.

Before commencement of the rockwall, a geo-textile filter layer shall be placed. The surface on which geo-textiles are placed shall be made relatively smooth, free of obstructions, depressions and soft pockets. Depressions shall be filled with sand. Placing of the geo-textile shall not be started until the underlying slope has been obtained. The geotextile shall be thermally bonded non-woven fabric constructed by needle punching staple fibers of polypropylene incorporating a minimum of 1% by weight active carbon black. The geotextile shall have the following properties.

Table 3.5 : Filter layer geotextile properties

Property	**Value**
Minimum mean water flow normal to the plane of the geotextile under 50mm head	40 l/sq.m/sec
Minimum mean coefficient of permeability	5×10^3 m/s
Mean maximum pore size 090	69 microns
Mean minimum tensile strength	55 kN/m
Mean minimum tensile extension	50%
Mean maximum Cone Drop perforation hole diameter	3 mm
Mean Minimum CBR puncture resistance	11000 N
Mean Minimum CBR puncture displacement	65 mm
Mean minimum thickness under 2 kPa	6.2 mm
Maximum thickness reduction under pressure increase from 2kPa to 200kPa	32%

Alternative geotextile materials which are used as a filter layer beneath rock armour may be acceptable. The proposed material and properties shall be approved by the client during prior to undertaking the works.

Holding the geo-textile in position shall be by ballasting with the filter rock. Pinning with steel pins or wooden pegs shall not be considered. The geo-textile shall be placed loosely without wrinkles or folds with the warp running normal to the coastline. The geo-textile will be laid in one piece over the required depth. Lapped joints shall not be permitted. Joining of geo-textile strips shall be achieved by stitching.

The quarry shall be identified from where sourcing of rock can be done. Availability of probable quantity and quality of rock shall be ascertained. All sorting and screening operations required for the production of rock in accordance with the specification shall be carried out at the quarry site. No rock shall be removed from the site until it has been sorted and/or screened and accepted as one of the specified rock gradings.

All rock to be sorted and screened shall be sound, compact, hard, dense, rough, durable rock, of good quality. The rock shall be free from seams, fissures, planes of weakness, blasting cracks and any other undesirable qualities. Rock placed in the works in bulk shall be transported and handled in such a manner to minimize segregation of the rocks and rocks placed in the works individually shall be transported and handled in such a manner as to minimize damage to the rocks and to ensure that the required rock grading is achieved.

Placement of rock in any section of the work to be constructed directly on the existing sea bed shall not commence until the pre-work survey drawings has been prepared. Filter layer material shall be placed to the positions and excavated depths indicated as per the construction drawings and in accordance with the approved method and sequence of construction.

3.7.4 Rock Base Construction

The components of the rock base are a 0.5 m filter layer and an armour layer. The rock base can be constructed using land-based or water based methods. However, it may be economical if a land based method is adopted as the construction equipment is readily available, less specialised construction equipment are required and availability of local labour compared to the construction of offshore structures. The usual equipment consists of backhoe excavators, front end loaders and trucks for rock delivery to the site. In addition, stones may be dumped from barges if this option is economical and the contractor has easy access to the seaward part of the structure.

The construction method should ensure that damage arising from wave attack during construction is minimal. The rock base requires large quantities of quarried rock, which may need to be supplied from distant quarries.

Placement of rock in any section of the work to be constructed directly on the existing sea bed shall not commence until the pre-construction survey drawings has been prepared. Filter layer material shall be placed to the positions and slopes indicated as per the construction drawings and in accordance with the approved method and sequence of construction. Filter layer material shall be dumped and tipped to the natural slope of the material and left untrimmed provided that the filter layer is built up to the dimensions shown on the drawings with the material specified for the armour layer overlying the filter layer and placed in accordance with the method for the overlying layer. The method of placement shall be such that all voids are filled to prevent subsequent surface collapse and settlement.

Techniques such as dumping from barges for the lower part and end tipping for the upper part may be used, providing the work is organized in such a way as to minimize segregation of the stone grading and to ensure the specified dimensions or weight per unit area.

Armor rock shall be placed individually. Individually placed quarried stone shall not be dropped or tipped into position, but shall be placed by piece into the structure to achieve a minimum 'three-point support' and be stable to the lines and levels shown on the drawings. Stones shall be tightly packed together so as to achieve the target specific gravity of stone placed of 2.6 t/m^3. The surface where the steel caisson is to be places shall be levelled with quarry run if required to get a uniform seating area for the caisson.

The front and side slopes of the rock base are steep and stones can be placed by backhoe excavators or cranes noting that cranes need a much stable work pad than a backhoe, which can crawl on an uneven stone layer. Stones delivered by dump trucks can also be placed by wire-rope cranes depending on its lifting capacity, boom length, boom angle and working radius.

It's imperative to try to eliminate material smaller than the minimum required to meet gradation. Care should be taken while placing the armour stones e.g. not breaking when dropped, running heavy equipment on the structure during construction and deliberately pushing small material onto the rock bases in order to build pads for the equipment to work on the structure. Good interlocking of carefully placed stones is essential for ensuring a long design life at the front and the edges of the rock base at the same time maintaining the porosity of the layer.

Continuous monitoring of the structure during its construction phase is a necessity. Sections have to be measured at the completion of each stone layer and visual control of the form and structure of the stone matrix has to be carried out. This is necessary to achieve the specified design and to make “as built” drawings as a reference point for further monitoring. It is also necessary to monitor the surrounding area during construction to ensure that every aspect is behaving as expected, such as sedimentation, scour etc.

Construction photographs of Borth, UK offshore rock reef is given below.

Figure 3.14. Dump truck transporting armour stone to construction site

Figure 3.15 Dumper truck carrying armour stone to the seaward end of the reef over a purpose built causeway and placement of armour rocks from the causeway using excavators

Figure 3.16 Excavator placing the armour stone upwards from the sea bed

Figure 3.17 Reef construction underway and dumper trucks delivering armour stones

Figure 3.18 Placing 3-6t armour rock tightly at low tide (UAE project picture)

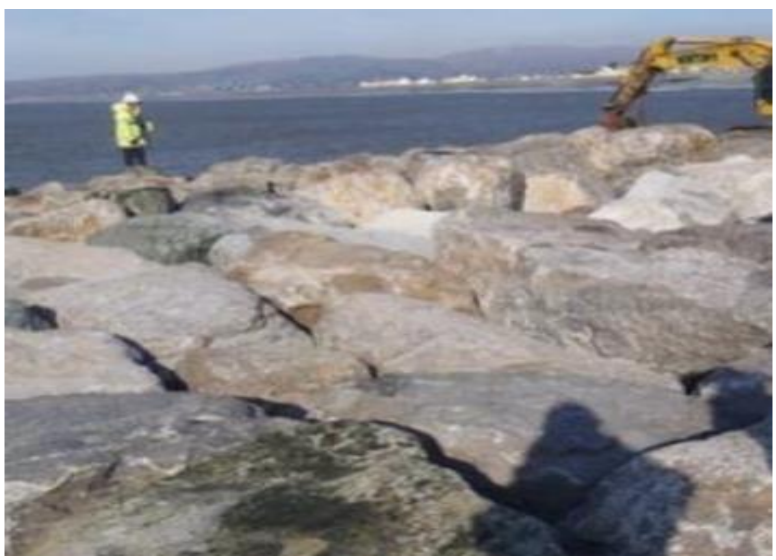


Figure 3.19 The paved placement of the Borth reef structure

Figure 3.20 Paved armour layer

Figure 3.21 Borth rock reef structures at low tide.

3.7.5 Concrete Wedge

The caisson can be built as a single unit or multiple units noting that the single caisson with slab thickness more than 150 mm will be difficult to float, if floating the caisson is required for placement. So an alternate option of construct the caisson in multiple units at site and slide into position with the help of crane, excavators and divers may be considered.

Each caisson unit is built starting with the slab. After the slab is ready, the construction of the upper part of the caissons begins, including: placing the reinforcement and pouring and vibrating the concrete. Once the caisson unit fabrication is completed, each caisson unit is lifted or slid with the help of crane and /or excavators to its final location. The caisson unit placement will start from the base to the front (inshore to offshore) due to the crane or heavy equipment requirement for the placement of the sections. As mentioned in Section 3.5.1 accurate placement of the caisson units are very important to ensure that there are no gaps between the caisson units to prevent water movement and to protect the rock base layer beneath it from scour by currents and wave action

3.7.6 Steel Wedge

The steel triangular caisson shall be fabricated 25 mm thick IS2062 grade steel sheets near seashore at a suitable location. The entire caisson can be built on land as a single unit and floated to the site for placement. The total weight of the caisson is in the order of 1200 t and has floating draft of 0.50 m. A fabrication yard shall be first constructed. It shall be made of longitudinal rail laid perpendicular to the shore and greased adequately to reduce the friction between steel and rail. These rails shall run into the sea to sufficient length so that the caisson comes into floating condition. The caisson shall be fabricated with its wider side at the landward end.

The steel caisson fabricated will have several chambers to adjust the floatation. Every chamber shall be tested with compressed air to see if it is leak proof. Each chamber shall be provided with a pipe to pump in water and another to pump out water. All tubes used for pumping out water shall be fitted with a foot valve. The pipes shall be sufficiently high to see that no water enters the chamber while in floating condition and when it is out in the sea. Sufficient number of mooring hooks/ rings and jacking points shall be provided on the caisson. All the welding should be from inside. The surface of the caisson shall be cleaned and painted with one coat of zinc rich primer coat and three coats of epoxy paint. Further sacrificial anodic protection is recommended to prevent or minimise corrosion of the steel plates

On completion of the caisson fabrication and painting it shall be pulled to sea using tugs and pushed from the shore using jacks. Once the caisson is in floating condition it shall be towed to the position using tugs.

The caisson will be sitting on a rock base and pre-deployment bed preparation and levelling works are to be undertaken. Pre-surveyed marker buoys should be placed at the placement location to ensure accurate placement of the caisson. All anchors or mooring points are to be clearly marked with marker buoys. After ensuring that the caisson is at the required position, the caisson shall be lowered to the bottom by filling the chambers with seawater through the pipes attached to the caisson top. The caisson needs to be sealed and the vertical pipes need to be removed on completion of the water filling.

3.7.7 Stability of partly completed works

The daily conditions of waves, wind and water levels that influence construction processes differ from the design conditions. General monthly wave climate and non-exceedance are presented in Table 2.1 and Table 2.3, for estimating site conditions for tender purposes, but the contractor needs to define the duration and sequencing of calmer periods and work activities.

3.7.8 Construction tolerances

The recommended tolerance for the finished profile of the reef structure at the time of acceptance by the Engineer is:

- Vertical placing tolerance +0.20 m and nil m
- Horizontal placing tolerance +1m and nil m.

To confirm the completed beach renourishment profiles are within tolerance, cross-sections are to be provided by the Contractor at a maximum spacing of 20 metres and extend from the existing cross-shore +5 m on the land to the -5 m water depth. Horizontal surveys are required along the cross-sections given in the construction drawings.

Surveys of the newly-completed structure are to be undertaken a 2 weeks after they are completed and allowing for the initial settlements

3.7.9 Final Surveying

At the completion of the reef construction, a detailed survey of the reef shape - both bathymetry and underwater video are required to ensure that the constructions are as per the design and tolerance requirements. Dive inspections are required to check the caisson is sitting on a stable rock platform.

3.8 South reef

The south reef is 200 m long and placed at the south end with the crest at 1 m above Chart Datum. The location of the southern offshore reef is at 300 m north of the pier (Figure 3.22). Any shore-parallel current that can pass between the reef and the beach can negate the wave induced current and flush the material from behind the structure. So the reef is designed as shorter sections of 25 m long separated by gaps of 3 m to allow over-topping water to flow back out to sea and reduce scour in the lee of the reef. The sections will mildly zig-zag to scatter the reflected waves. The reef can be constructed using steel caissons, rock or geotextiles. The plan view and cross-sections of steel caisson are given in Figure 3.23 and Figure 2.24 and reef specifications are given in Table 3.6

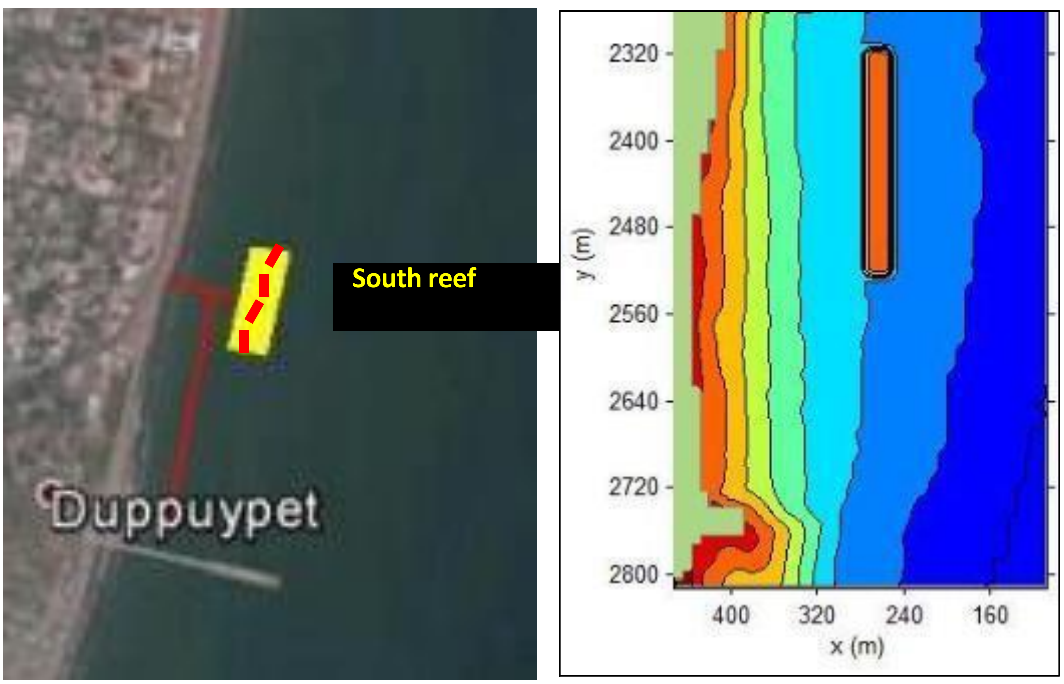


Figure 3.22 The location of the southern offshore reef, at 300 m north of the pier (left) and a close up view of the reef on exisitng bathymetry.

Table 3.6 Southern reef specifications

Reef Information	
Reef orientation	10° east of north
Reef length	200 m
Reef width at base	38 m
Reef height above seabed	5 m
Reef volume	12,000 m^3
Reef footprint	7,600 m^2
Reef crest elevation	0-1 m above CD
Depth at reef	4 m

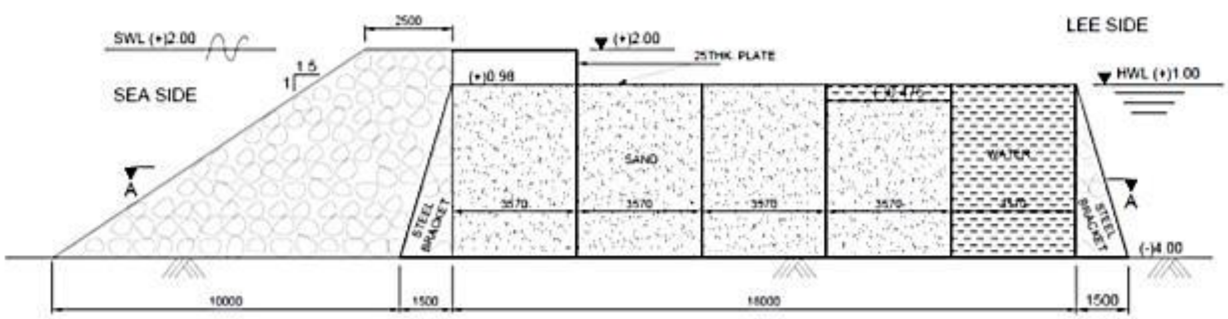


Figure 3.23. South reef steel caisson plan view

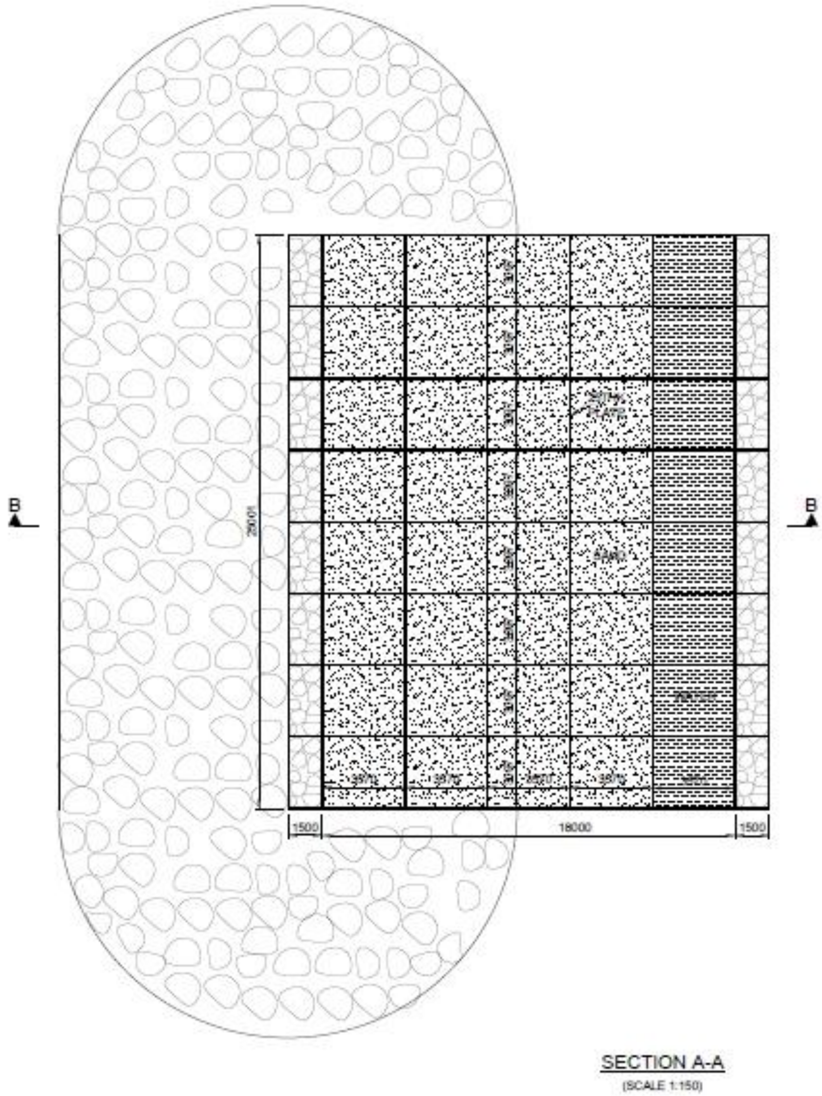


Figure 3.24. South reef steel caisson cross-section

3.9 South reef construction

The construction methodology of the south reef depends on the material used for the construction. If steel or rock is selected, the construction methodology of the wedge reef can be considered. This section discusses a geotextile construction methodology.

If geotextile is used, the reef will consist of 194 geobags in a 4 layer structure. To achieve the desired crest elevation, the design consists of a 4-3-2-1 layout of the containers as seen in Figure 3.25. The base layer consists of 50 x 28 m long containers with a theoretical height of 1.6 m. The2nd, 3rd and 4th (top) layer containers are 21 m, 16 m and 11.5 m long respectively. All the containers are 4 m wide and 1.6 m high, commonly known as "T2" geobags (Table 3.7). The estimated volume of the reef is 12,856 cu m. Given the short weather window for marine underwater works, we estimate that construction will take 2-3 years to complete.

Table 3.7. Geotube dimensions and volumes for a 4-layer reef of 200 m long

Reef Layer	Height (m)	Width (m)	x-sectional area (sq m)	Length (m)	Volume/tube (cu m)	Total volume (cu m)	Total tubes (No.)
4 (top)	1.6	4	5.5	11.5	63	2,973	47
3	1.6	4	5.5	16	88	4,224	48
2	1.6	4	5.5	21	116	5,660	49
1 (bottom)	1.6	4	5.5	28	154	7,700	50
						12,856	194

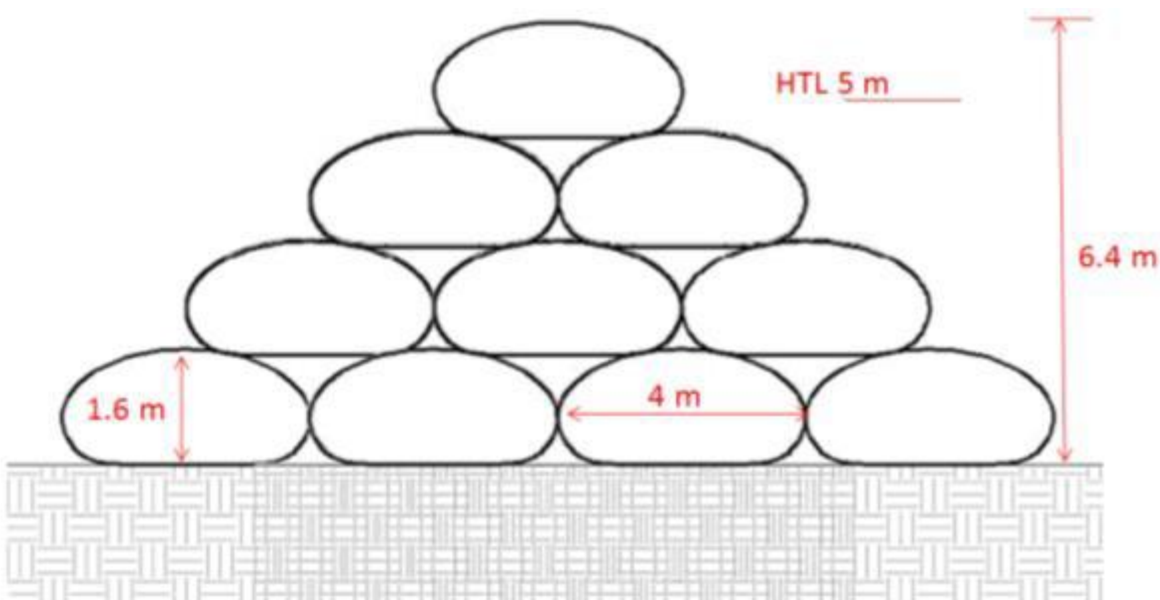


Figure3.25. 4-layers geotextile reef side view. Each geotube is 4 m wide and 1.6 m high.

3.9.1 Reef construction using geotextile megacontainers

Geotextile megacontainer placement and filling can be either land based or sea based depending on the site conditions, access, sand availability, filling method etc. Prior to the construction, the site must be prepared such that there is no debris and the filling area is level and firm. Failure to ensure a level and firm construction area may lead to damage or instability.

Geotextile mega container installations can be sensitive to climatic conditions including tides, waves, rain and wind. Tidal variations may influence the availability of fill material, the ability to place and the area available to work and store raw materials and equipment. For safety reasons, strong or severe wave actions can have an effect on the ability to work within an exposed coastal region. Rain and wind can present hazardous situations in and around the work site, particularly where electricity is present. All of the above factors must be taken into account when planning an installation.

3.9.2 Sea-Based Filling

Figure 3.26 shows an overview of the sea-based filling operation used for the Kovalam reef construction. A dredge pump is located at the end of a hydraulically controlled digger arm and is lowered to the sea bed. Water jets on the pump agitate the sand beneath the pump, causing it to go into suspension where it is sucked into the pipeline and pumped to the reef. The outlet of the dredge line is controlled by a diver who must insert the dredge line into the filling port on the geotextile container and monitor the filling process. Problems encountered with the sea based operation include rough seas and barge stability as well as difficulties inserting the dredge line into the filling ports.

Figure3.26. Geotextile megacontainer pre-deployment preparations ,placement and underwater filling

3.9.3 Land-Based Filling

A land-based filling method was used for the construction of the Mirya reef, Ratnagiri project and this is used as an example in the following section. While this technique does not have the problems associated with working on the water, it does present additional challenges. Primarily the need for large amounts of water to be mixed with the sand to form a slurry that can be pumped along the dredge line.

The sand-pumping system used at Mirya, Ratnagiri. Figure 3.27 shows the water intake lines that are connected to 6-inch diesel powered water pumps. These pumps are used to draw water from the ocean up a pumping site on land and into a sump where the sand/water slurry is mixed. The sand is delivered into the sump by a variable speed conveyor belt visible in Figure 3.27 which is loaded using a front-end loader and the mixture is agitated by the incoming water supply. A large dredge pump is located inside the sump and pumps the sand/water slurry out through the pipeline and out to the reef site. The maximum pumping distance in this example is over 250 m- pushing the limit for effective delivery with the available pump and pipeline dimensions. Larger pumps would be able to pump larger distances or through larger pipes.

Figure 3.27. Mirya reef construction: land based sand slurry pumping

3.9.4 DEPLOYMENT

Mark Out Location and Alignment: In all applications, this process is critical for successful installation. Marking out and installing anchor points will help ensure the mega container is in the right place and is aligned. There are several options for anchor points depending on the application and these range from fence posts to concrete blocks. The selection of the most suitable option is based on the available equipment.

1 Install 2 leading anchor points each offset 1.5m from the container's centre line.

2 Install the anchor points at 5m centre's along the length of the container, offset 4m from the centre line.

3 Install the 2 end anchor points each offset 1m from the centre line.

Connect Dredge Line : When the mega container is in position the dredge line must be securely fastened to the fill ports. This connection must be secure as the volume of material entering the tube and the force created by the dredge is significant.

1 Pull filling trunk out of first fill port of the mega container.

2 Insert dredge line into mega container; ensure pipe extends beyond the end of the filling trunk.

3 Tie off the trunk to the dredge line, a short straight flanged section or elbow is ideal for the inlet section as it allows the trunk to be locked in place behind the flange. Locking the dredge line onto the port is achieved using a ratchet tie down clamp.

4 The dredge line should be positioned so that the flow is directed down the length and along the top of the mega container. If the dredge line is incorrectly positioned the mega container will tend to roll towards the direction of flow.

FILLING: Filling of the mega container will be by pumping sand slurry mixture. The dredge should be capable of delivering 20 to 30% solids. However factors such as grading of dredge material and pumping distance will affect the solids delivery rate. Depending on the environmental conditions, treatment of the pass through material may be required to prevent turbidity or contamination.

Before pumping the slurry, allow the mega container to inflate and discharge water through the outlet ports. This will help the slurry pumping as well as ensure maximum height of the containers is not exceeded during the filling process. The discharge ports must not be closed during filling as this may result in excessive pressure build up within the mega container and possible rupture of the seams. If required, discharge excess pressure through a Y piece in the dredge line. The divers may have to move the dredge line to secondary filling ports on longer tubes or if the dredge is incapable of supplying consistently high volumes of sand/water mix.

Filling of a standard 20 m mega container should take between 6 and 8 hours and depending on dredge and fill material quality. Coarse fill material will result in faster fill times while moist and fine material will result in longer periods for filling.

During the slurry pumping, divers must measure level of fill material within the mega container by pushing firmly against the side or top of the container. The mega container is full when solid and unyielding under foot. Once filling is completed, remove anchors and cut off locating ropes as close to the mega container as possible. Care must be taken not to damage the container during this operation.

The final step in the process is to seal the mega container to prevent the material escaping, and to provide a neat finish. It is recommended to fill any depressions within the container. Once the container is full and ready to cap, roll up and tie off the filling trunks with cable ties. Then push trunks back into the mega container and lace the filling port closed using cord through the holes in the container fabric. If any cover is provided by the manufacturer, place the supplied cover over port, punch first locating hole in mega container using sharpened screw driver and use screw supplied to fix in place.

Table 3.8 Geotextile shall be a composite polyester and propylene material. The geotextile shall have the following properties

Property	Test Method	Unit	Recommended Geotextile
Physical (MARV)[1]			
Polymer			UV Stabilised Polyester + Polypropylene
Mass - Base	AS3706.1	g/m2	1,080
Mass - Coating	AS3706.1	g/m2	800
Thickness @ 2kPa pressure	AS3706.1 (ASTM D5199)	Mm	10.5
Mechanical (MARV)[1]			
CBR Puncture Strength	AS3706.4 (ASTM D5199)	N	10,000
CBR Elongation		Mm	50
Wide Width Tensile Strength XD/MD	AS3706.2 (ISO 10319)	kN/m	45/90
Wide Width Tensile Elongation (weakest direction)	AS3706.2 (ISO 10319)	%	88
Trapezoidal Tear Strength (weakest direction)	AS3706.3 (ASTM D 4533)	N	900
Seam Strength Efficiency XD/MD		%	>70/>90
Hydraulic (TYPICAL)			
Water Permeability	AS3706.9	m/s	3.0×10^{-4}
Flow Rate	AS3706.9	$l/m^2/s$	27
Pore Size	AS3706.7	µm	<75
Fines retention (Hydrodynamic)	NFG 38.017 (Modified)	%	95
Durability (MARV)[1]			
Resistance to Weathering (UV resistance after 500hrs exposure)	AS3706.11 (ASTM D4355)	%	75
Abrasion Resistance (Strength retained after 80,000 revolutions)	BAW Rotating Drum	%	75

[1]MARV : Minimum Average Roll Value

1. All geotubes must be sand coloured nonwoven composite 2000 gsm, UV Stabilised Polyester + UV stabilised
2. Polypropylene (PP) fibres shall be ;
    a. 100 denier staple fibre
    b. homogenously needle punched into the polyester geotextile substrate to form an integrally formed dual layer
    c. There shall be strong physical bonding between the upper and lower layers formed by needle punching
    d. Mass of the 100 denier PP layer shall be not less than 800 gsm
3. Manufacturer shall provide evidence of UV stabilisers added to polypropylene fibre such that they meet the maximum, extra out-door level 4 requirements
4. Suppliers shall provide full details of working seam and performance values in both machine direction (MD) and cross machine direction (XD)

3.9.5 Durability

The key factors influencing the long term durability of the geotextile sand containers are (i) incidental damage (from driftwood or boat damage) (ii) vandalism (knife cuts and punctures) (iii) seam failure, (iv) under filling or over filling of the containers. Examples of geotextile megacontainer damage photographs are given below (Figures 3.28 - 3.33).

Figure 3.28 Impacts due to wave reflection: toe scour, undermining, differential settlement

Figure 3.29 Failure due to Undermining Most common failure mode


Figure 3.30 Dislodged and damaged containers, Ullal, Karnataka


Figure 3.31 Punctured woven geotextile - Candolim, Goa

Figure 3.32 Under-filling could cause sand movement inside the containers under wave action and over-filling could affect the seam efficiency

Figure 3.33 Effective closure of fill ports is critical to the long-term integrity of the structure, as experience had shown that ports, which allow the loss of fill material results in deflation of the containers.

3.10 Beach Nourishment

3.10.1 Length of beach to be nourished

The planned beach nourishment at Puducherry is to extend over about 1.5 km using 450,000 cu m of dredged sand. The beach's length to be nourished under the beach nourishment programme is about 1 km between the south reef and the wedge reef (Figure 1.1). If nourishment commences between February and August, it is recommended to place the sand between the south reef and the statue of Gandhi. However, if the nourishment works starts between October and December, it is recommended to start the sand placement to the north of the Gandhi Statue area. The recommended annul maintenance nourishment is 50,000 cu m per year.

3.10.2 Placement of Nourished Sediment on the Beach Profile

Various design schemes have been used for the placement of nourished sand on the beach : (1) dune nourishment - placing all of the sand as a dune backing the beach, (2) dry beach nourishment - using the nourished sand to build a wider and higher berm above the waterline (3) profile nourishment - distributing the added sand over the entire beach profile, (4) nearshore bar nourishment - placing the sand in the shallow offshore as an artificial bar (Figure 3.34). The selected design depends in large part on the location of the source material and the method of delivery to the beach. If the borrow area is on land and the sand is transported by trucks to the beach, placement on the berm or in a dune is generally most economical. Given that for the present project the sand source is on land, the recommended nourishment method for this project is dry beach nourishment, which is a very common approach

In the dry beach method, sand is placed on the dry portion of the beach and near the waterline, and results in an immediate increase in beach width available for recreation (Figure 3.35). Once the sand is placed on the beach, waves and currents redistribute the material offshore and alongshore until a stable profile configuration is achieved. The nourished beach may take weeks to several months to reach the equilibrium condition depending on local conditions.

Note that as explained in the Task 1 report, equilibrating process results in a substantial slumping and narrowing of the initial dry beach width. With profile adjustments to reach an equilibrium condition, the general public may perceive the narrowing of the initial dry beach width as a sign of failure of the project. Therefore there is a need for public education at the onset of the project so that the public understands that some initial alongshore and offshore sediment movement and erosion of the berm are expected. Also, the public needs to recognise that so long as the sand remains in the littoral zone within the envelope of beach profile changes, the sand has not actually been "lost”. Although the profile adjustment will in most cases result in shoreline recession, the material will still be present in the active beach profile; much of it will be in the offshore bar and on the berm. As explained in Task 2 Part I, the south reef and the nearshore wedge reef are expected to reduce alongshore and offshore sediment transport from the nourishment site.

For the annual maintenance nourishment, it is recommended to place the sand in the nearshore (2-3 m water depth) off Gandhi Statue as the disposal sediment would be active and move quickly onto the sub-aerial beach. The nearshore bar nourishment in effect immediately introduces the sand into the nearshore zone of active profile changes where the nourished material can be readily incorporated into the overall beach profile.

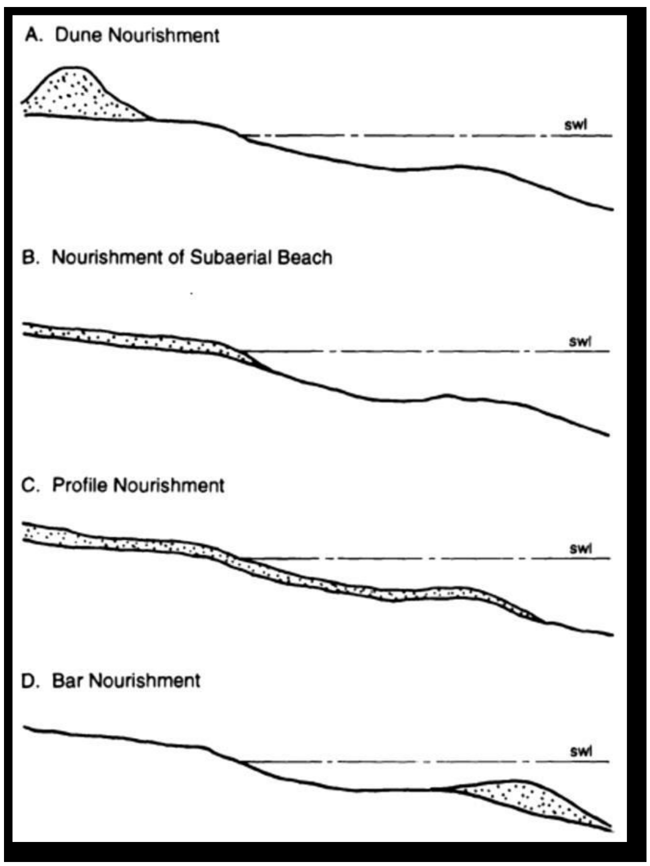


Figure 3.34 Schematic representation of a series of beach-fill profile designs used in nourishment projects, ranging from placing the sand in dunes backing the beach to its placement in the offshore as mound or bar (source : Komar, 1988)

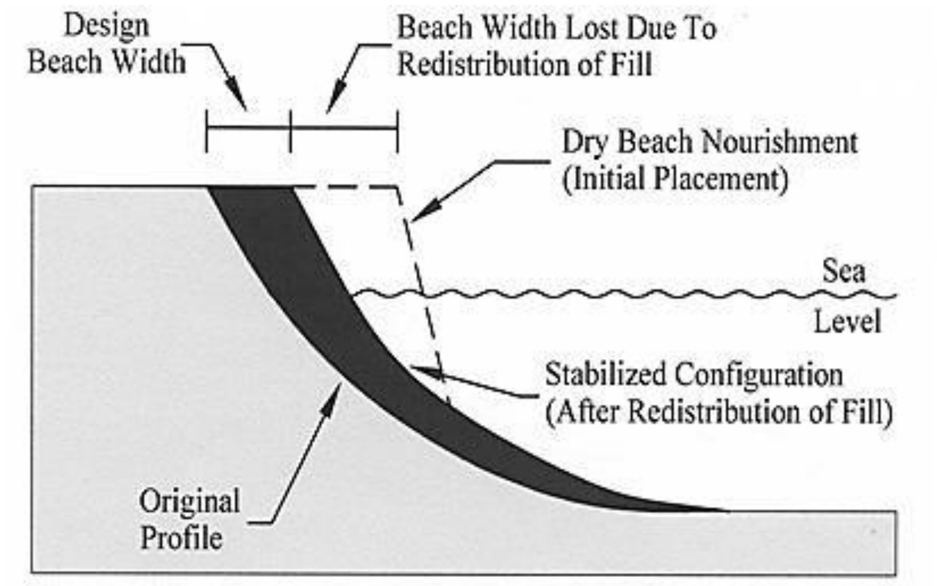


Figure 3.35 Schematic representation dry beach nourishment (Source : USACE, 1992)

3.11 Transport of borrow sediment to the nourishment site

Generally, there are two methods of transport and placement of borrow material for a beach nourishment: hydraulic and dry methods. Hydraulic methods are generally used for material obtained from marine-based sources and dry methods for material obtained from land-based sources though hydraulic method is employed for land based source depending on the site conditions. For the Puducherry nourishment project, the borrow sand stockpiled on land may be trucked to the site and placement on the dry beach is obviously the most economical and efficient method. However, for sand from any new dredging of the port channels, direct pumping to the site in slurry form may be the preferred option. The sand delivered to the shore either by truck or direct pumping then needs to be groomed using earth moving equipment to the desired construction profile (Figure 3.36).

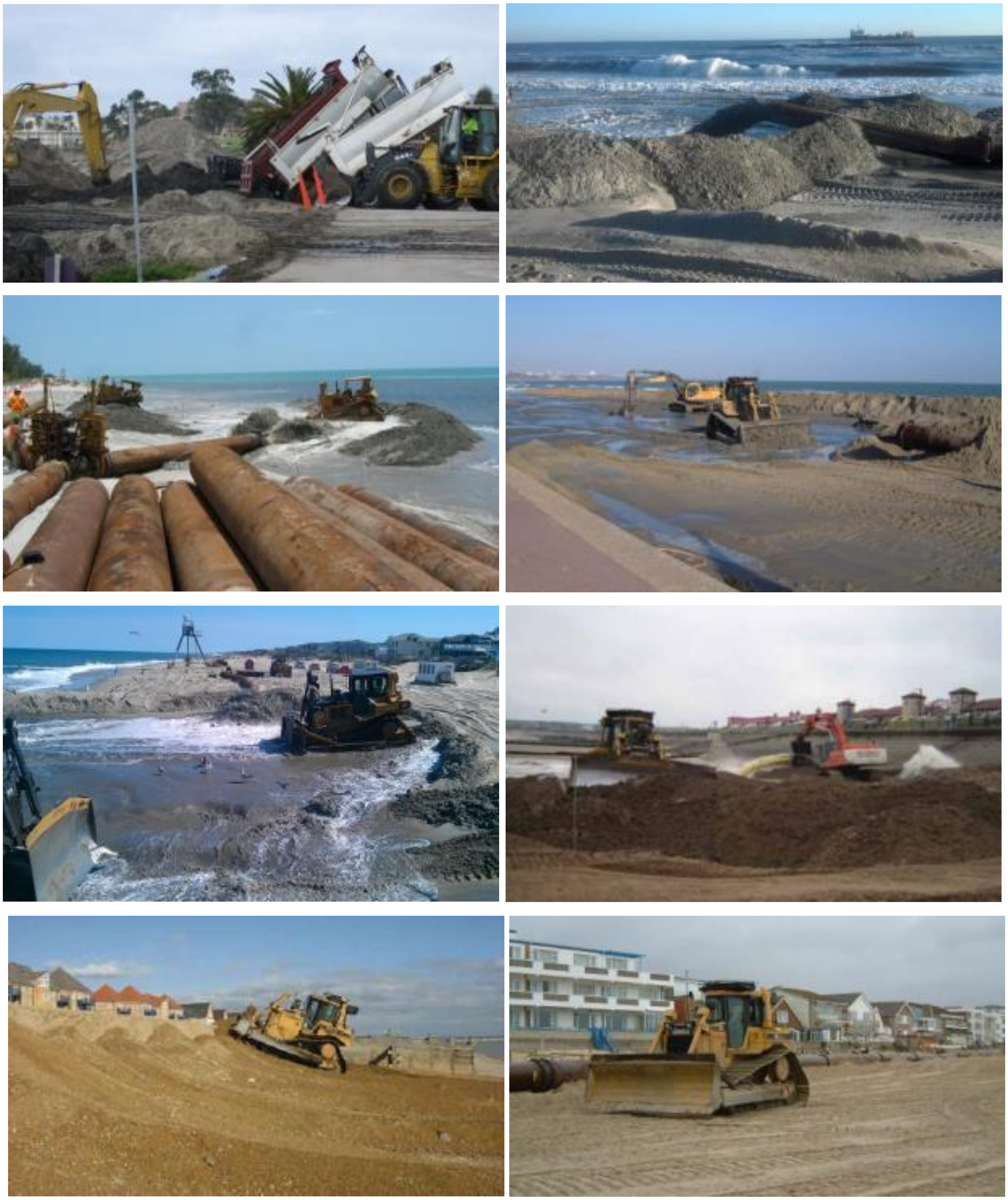


Fig 3.36 Photographs showing various activities involved in beach nourishment works

3.11.1 Monitoring

Following construction, the beach nourishment needs to be monitored to evaluate the project performance and to regularly assess the condition of the nourishment. These include shoreline and berm positions, total volume, and the response of the beach to a storm. Bathymetric and beach profile surveys, beach sediment sampling, satellite imageries, and wave and water level monitoring would provide an accurate and objective measure of the nourishment project's response. Without physical monitoring data, it is difficult to estimate how well the project is performing in comparison to the design. Most monitoring programs involve an early phase of more intensive data collection of bathymetric surveys, beach profiles, sediment and marine ecology to evaluate project performance. After the project performance is established, data collection is scaled back to focus on monitoring project condition.

Bathymetric surveys need to cover the areas between the port's north breakwater to at least 5 km north. Beach profile surveys need to extend from the crest of the present seawall, to across the entire active zone of sediment transport, which is about 6 m for Puducherry. To get an adequate resolution of this beach nourishment project, it is recommended a longshore profile spacing of 100 m for the nourishment area between the 2 reefs and at 250 m alongshore spacing for the remaining beach.

The bathymetric and beach profile data should be acquired to adequately define beach and sea-bottom slopes, changes in slopes, and prominent morphologic features, such as berms, bars, and shoals.

For the project monitoring purpose, a full pre-project (baseline) survey should be undertaken, followed by a post-nourishment survey. Surveys are then performed twice a year, typically at the end of north east monsoon (February) and southwest monsoon (September) to determine the full excursion of seasonal changes in the subaerial beach width and volume. After the first 3 years of monitoring, the survey might be reduced to one per year, the September survey, when low-wave conditions are prevalent.

Beach profile surveys need to be referenced vertically and horizontally to a permanent marker. So, for the monitoring purpose, it is essential to install permanent survey control points or a survey baseline for individual transects and should be located landward of the mean high water line. The elevation of these survey control points need to be referenced to the datum used for the beach profile survey, preferably the chart datum and its location is tied into the UTM coordinate system or latitude/longitude system. The profile surveys should be undertaken using an electronic Total Station, or any sophisticated survey instrument to provide a high level of horizontal and vertical accuracy.

The monitoring plans, if fully implemented, should provide valuable information to evaluate the effectiveness of the project impacts as well as to determine when any additional remedial actions may need to be considered.

3.12 Construction Schedule

Construction of the reefs is scheduled to commence in early 2017 and be completed in 12 months. It should be noted that this timescale is indicative only and is for the purpose of this report. The actual timing of the works following completion of nearshore wedge reef will depend on a number of variables, including time to complete the offshore works before monsoon, the fund availability for the south reef, and sand availability for the beach nourishment

The contractor will also take responsibility for ensuring that all the works are in keeping with the Health and Safety requirements under the Construction

The Wedge reef construction will begin in early 2017 and continue until complete. A 5 months (February - June 2017) construction period is anticipated. The construction of the wedge reef requires the use of plant in the water and may require the use of floating plant and divers. January sea conditions in Puducherry make it unsafe to undertake such operations later than the end of January. Construction of the wedge reef is not likely to start until February 2017. However, stockpiling of materials at the work area or other suitable sites can commence during December 2016 - January 2017 so that works can begin as soon as weather conditions allow.

Beach nourishment could take place following construction of the south reef.

Bill of Quantities

The following tables give a summary of the cost estimate for various reef options. A detailed cost analysis will be provided on finalisation of the reef option, material selection and construction methodology

A NORTH WEDGE REEF

A1 North Wedge Reef: Concrete, Single Unit, slab thickness 150 mm

Item	Amount (US$)	Amount (INR)
Construction of dry dock	2,543,777	167,889,283
Casting of concrete caisson (single unit, slab thickness 150 mm)	389,807	25,727,244
Towing and placing of caisson at site	757,576	50,000,000
Rock base	1,194,377	78,828,858
Total	**4,885,536**	**322,445,386**

A2 North Wedge Reef : Concrete, Multiple Units, slab thickness 200 mm

Item	Amount (US$)	Amount (INR)
Casting of concrete caisson (multiple units, slab thickness 200 mm)	500,000	33,500,000
Towing and placing of caisson at site	900,000	60,300,000
Rock base	1,194,377	80,023,235
Total	**2,594,377**	**173,823,235**

A3 North Wedge Reef : Steel, single unit, steel thickness 25 mm

Item	Amount (US$)	Amount (INR)
Casting of steel caisson (single unit, 25 mm thick steel)	1,235,606	82,785,606
Towing and placing of caisson at site	100,000	6,700,000
Rock base	1,194,377	80,023,235
Total	**2,529,983**	**169,508,841**

B SOUTHERN REEF

B1 Steel Caisson with Rock Scour Protection

Item	Quantity	Unit	Rate	Amount	
				US$	INR
Steel sheet					
Supplying and casting of steel caisson	2,320	T	70,000	2,423,881	162,400,000
Rock structure					
Supplying and instaling at site 1.5 T rock for armour	9,196	T	1,095	150,285	10,069,073
Supplying and instaling at site 500 kg rock for core	11,872	T	1,064	188,534	12,631,808
Launching of caissons and seating in position including filling with sand and placing scour protection rocks	8	No.	1,500,000	179,104	12,000,000
Surveys, reporting etc	10	No.	90,000	13,433	900,000
Mobilisation, demobilisation	1	Ls	750,000	11,194	750,000
Total				**2,966,431**	**198,750,881**

B2 Geotextile Reef : 4 layers

Item	Unit	Amount ($)	Amount (INR)
Non woven geotextile containers	194 Nos	467,500	31,322,500
Geomat (5 x 100 m roll)	15	43,209	2,895,000
Geocontainer placement and filling	12,856	2,226,818	149,196,781
Surveys, reporting etc	Lump sum	40,000	2,680,000
Mobilisation, demobilisation	Lump sum	50,000	3,350,000
Shipping, Taxes, import duty etc	1	312,000	20,904,000
TOTAL		**3,139,527**	**210,348,281**

PUDUCHERRY BEACH RESTORATION PROJECT				
PART A- CONSTRUCTION OF DRY DOCK				
Sl No	**Reference**	**Description**	**Unit**	**Quantity**
1		Supply and driving of straight web sheet pile	**T**	960
2	12.1.1B.3;MORTH	Earth work in excavation for foundation of cum structures as per drawings and technical specifications, including setting out, construction of shoring and bracing, removal of stumps and other deleterious matter, dressing of sides and bottom and backfilling with approved material - Oridnary Soil - Mechanical Means - above 6 metres depth including the cost of labour, hire charge of hydraulic excavator and overhead charges; in the proposed dry dock area	Cum	81783.75
		All kinds of soil.		
3		Dewatering	**Ls**	_
4		Providing and laying in position ready mixed plain cement concrete, using fly ash and cement content as per approved design mix and manufactured in fully automatic batching plant and transported to site of work in transit mixer for all leads, having continuous agitated mixer, manufactured as per mix design of specified grade for plain cement concrete work, including pumping of R.M.C. from transit mixer to site of laying and curing, excluding the cost of centering, shuttering and finishing, including cost of curing, admixtures in recommended proportions as per IS : 9103 to accelerate/ retard setting of concrete, improve workability without impairing strength and durability as per direction of the Engineer - in - charge.		
		All works up to plinth level:		
	4.19.1.: DAR 2014	M-15 grade plain cement concrete (cement content considered @ 240 kg/cum).	Cum	1020
5		Excavation in the frontage for releasing the caisson	Cum	12250
6		Removal of sheet pile	T	960

7	2.25-2.26.1 DAR 2014	Refilling the work area-Filling available excavated earth (excluding rock) in trenches, plinth, sides of foundations etc. in layers not exceeding 20cm in depth, consolidating each deposited layer by ramming and watering, lead up to 50 m and lift up to 1.5 m.		
			Cum	81783.75
PART B- CASTING OF CAISSON				

1	5.33.1;DAR 2014	Providing and laying in position machine batched and machine mixed design mix M-25 grade cement concrete for reinforced cement concrete work, using cement content as per approved design mix,ncluding pumping of concrete to site of laying but iexcluding the cost of centering, shuttering, finishing and reinforcement, including admixtures in recommended proportions as per IS: 9103 to accelerate,retard setting of concrete, improve workability without impairing strength and durability as per directionof Engineer-in-charge.		
1.a	5.34.3:DAR 2014	Providing M-40 grade concrete instead of M-25 grade BMC/RMC.(Note : Cement content considered in M-40 is @ 360 kg/cum)	m3	1105.43
2	5.22.6; DAR 2014	Steel reinforcement for R.C.C. work including straightening, cutting, bending, placing in position and binding all complete upto plinth level		
		Thermo-Mechanically Treated bars		
		For columns@250kg/m3	kg	4662
		for beam @200kg/m3	kg	76955.6
		for slab @90kg/m3	kg	63234.93
				144852.5
3		Centering and shuttering including strutting, propping, etc., and removal of form for:		
3.a	5.9.6 DAR 2014	Columns, Pillars, Piers, Abutments, Posts and Struts	m2	248.64
3.b.	5.9.3 DAR 2014	Suspended floors, roofs, landings, balconies and access platform	m2	3153.892
3.c.	5.9.5 DAR 2014	Lintels, beams, plinth beams, girders, bressumers and cantilevers	m2	1949.612

PART C- TOWING AND PLACING OF CAISSON AT SITE				
		Ls		
PART D-ROCK STRUCTURE OF REEF				

1	2.25-2.26.1 DAR 2014	Filling available excavated earth (excluding rock) in trenches, plinth, sides of foundations etc. in layers not exceeding 20cm in depth, consolidating each deposited layer by ramming and watering, lead up to 50 m and lift up to 1.5 m.		
		Extra for every additional lift of 1.5 m or part thereof in excavation /banking excavated or stacked materials.		
		All kinds of soil.	Cum	9536.63
2	65.29 ; HE D SoR	Supplying granite quarry run from approved quarry to sorting site by lorry/ tipper, and sorting the stones into 5 kg to 10 kg category stones of approved quality with specific gravity ranging from 2.65 to 2.8 for forming the filter layer of breakwater inclusive of cost of stones, hire of lorries and machineries, labour charges required at quarry and at sorting place and the measurements of categorized stones after proper sorting are taken on weigh bridge installed at site at the cost of the contractor with approved software having printouts using contractors supplied papers, stationeries and conveyed to the approved alignment of the breakwater including conveyance from sorting platform and dumping stones using tippler (3.5x2.5 sqm size) or any suitable methods installed at site on a moving crane having a capaciaty of not less than 20T and placing the stones at sea bed in uniform layer of design thickness for forming filter layer of breakwater as per the approved drawing and design and inspecting the profile once in a week including hire and operational charges of T & P, tippler, crane and all incidental charges etc. complete as per the direction of departmental officers at site		
		for bedding layer of 0.50 thick	T	8212.271

3	65.3 ; HED SoR	Supplying granite blasted rock from approved quarry to sorting site by lorry/ tipper, and sorting the stones into 10kg to 200kg category stones of approved quality having specific gravity ranging from 2.65 to 2.8 for forming the core of rubble mound breakwater inclusive of cost of stones, hire of lorries and machineries, labour charges required at quarry and at sorting place and measurements of categorized stones after proper sorting are taken on weigh bridge installed at site at the cost of the contractor with rpinprgovDedepsaorftwaerenthaving printouts using contractors supplied papers, stationeries and conveyed to the approved alignment of the breakwater and rehandling and forming the core of the break water to the lines and levels as per the approved drawings and inspecting the profile once in a week with tolerance of +/- 20cm in final levels including all cost and labour charges, hire and operational charges of light crane and cost of spalls/ quarry muck and labour for forming the roadway for movement of lorries/ tippers / cranes etc including all incidental charges etc complete as per the direction of departmental officers at site		
		for the toe layer	T	2803.20

4	65.31; HED SoR	Supplying blasted rock from approved quarry to sorting site and sorting the stones into 300kg to500kg category stones of approved quality having specific gravity ranges from 2.65 to 2.8for forming the secondary armour and berm of rubble mount breakwater inclusive of cost of stones, hire of lorries and machineries, labour charges required at quarry and at sorting place and the measurements of catagorised stones after proper sorting are taken on weigh bridge installed at site at the cost of the contractor with approved software having printouts using contractors supplied papers, stationeries and conveyed to the approved alignment of the breakwater and forming the secondary armour and berm of the breakwater to the lines and levels as per approved drawings with tolerance of +/- 20 cm in final levels including all cost and labour charges, hire and operational charges of mobile crane and excavator, rehandling, placing and packing and using mobile crane, inspecting the profile once in a week and cost of spalls/quarry muck and hire of machineries forEngineefroirnmgi nDgetphaertrmo aednwt ay for movement of lorries/tippers/cranes etc including all incidental charges etc complete as per the direction of departmental officers at site	T	48739.86

5	65.32; HED SoR	Supplying blasted rock from approved quarry to sorting site and sorting the stones into 500kg and above category stones of approved quality having specific gravity ranges from 2.65 to 2.8 for forming the primary armour and berm of rubble mount breakwater inclusive of cost of stones, hire of lorries and machineries, labour charges required at quarry and at sorting place and the measurements of catagorised stones after proper sorting are taken on weigh bridge installed at site at the cost of the contractor with approved software having printouts using contractors supplied papers, stationeries and conveyed to the approved alignment of the breakwater and forming the primary armour of the breakwater to the lines and levels as per approved drawings with tolerance of +/- 20 cm in final levels including all cost and labour charges, hire and operational charges of mobile crane and excavator, rehandling, placing and packing and using mobile crane, inspecting the profile once in a week and cost of spalls/quarry muck and hire of machineries for forming the roadway for movement of lorries/tippers/cranes etc including all incidental charges etc complete as per the direction of departmental officers at site		
		For core in the reef	T	12170.29

References

BSI (1991) Maritime Structures - Part 7 : Guide to the Design and Construction of Breakwaters (BS 6349:Part 7 : 1991). British Standards Institution, London, 88p.

CIRIA (1991) Manual on the Use of Rock in Coastal and Shoreline Engineering. Construction Industry Research and Information Association, United Kingdom, 907p.

Van der Meer, J.W. (1990). Rubble Mounds - Recent Modifications, Handbook of Coastal and Ocean Engineering, Volume 1, edited by J.B. Herbich, Gulf Publishing Company, Houston, pp. 883-894.

Appendix 1: Determination of size of rock : Hudson and Van der Meer formulae

This appendix discusses the Hudson Formula and the Van der Meer Formulae for calculating the size of rock armour of rubble mound structures

A. Hudson Formula

The Hudson formula was derived from a series of regular wave tests in a laboratory with scaled breakwaters. The formula is given by :

Where W = weight of armour unit (N)

H = Design wave height at the structure (m) K_D

= Dimensionless stability coefficient

α = Slope angle of structure (radians)

ρ_r = Mass density of armour

g = Acceleration due to gravity (9.81 m/s^2)

Δ = Relative mass density of armour = $(\rho_r / \rho_w) - 1$ ρ_w

= Mass density of seawater

For non-breaking wave conditions, the recommended design wave height is $H_{1/10}$ at the site of the structure. For conditions where $H_{1/10}$ will break before reaching the structure, the wave height used in design should be the breaking wave height or the significant wave height, whichever has the more severe effect (BSI, 1991).

The number of rock units per cubic metre is estimated using:

$$— \quad (\quad —)(—)^{-}$$

Where Nr = number of rock units A

= Area applicable, in m^2

n = Number of layers of armour k_Δ =

Layer coefficient

P = Porosity of the armour layer

W_r = Unit weight of the material of the armour block W

= Unit weight of water

$$- \qquad (\quad -)(-)^{-}$$

$$(-)^{-}$$

B. Van der Meer Formulae

For estimation of rock size of the rock platform software developed by Dr P.V. Chandramohan for breakwater design, following Van der Meer procedure for shallow water breakwater design has been used (Chandramohan personal communication).

Van der Meer derived two formulae for submerged rock platforms under plunging and surging waves. These formulae take account of the influence of wave period, storm duration, armour grading, spectrum shape, groupiness of waves, core permeability and damage level on rock armour, and therefore they are described as practical design formulae for rock armour.

In shallow water conditions the wave load changes. In order to take into account the effect of the changed wave distribution, the stability of the armour layer would in the depth limited conditions be better described by using the 2 per cent wave height H 2%, than by the significant wave height, H_s (Van der Meer, 1988).. These results indicated that if the reef is located in relatively shallow water and that if the wave height distribution is truncated by breaking, the 2% value of the wave height exceedance curve gives the best agreement with results showing a

Rayleigh distribution (Van der Meer, 1990). He is assuming that the largest waves cause most damage and that by correctly truncating the wave height distribution, a smaller armour size can be justified. The modified Van der Meer formulae for shallow water conditions given in CIRIA (1991), are :

For plunging waves ($\xi_{s-1,0} < \xi_{cr}$):

$$\frac{}{} \qquad \left(\frac{}{\sqrt{}}\right) \quad \left(\frac{}{}\right)\left(\qquad\right)$$

For surging waves ($\xi_{s-1,0} \geq \xi_{cr}$)

$$\frac{}{} \qquad \left(\frac{}{\sqrt{}}\right) \quad \left(\frac{}{}\right)\sqrt{\qquad}\left(\qquad\right)$$

where

N = number of incident waves at the toe, which depends on the wave conditions

H_s = significant wave height (m) of the incident waves at the toe of the structure

$H_{2\%}$ = wave height exceeded by 2 per cent of the incident waves at the toe (m)

$\xi_{s-1,0}$ = Surf similarity parameter using the energy wave period $T_{m-1,0}$ (s) from time-domain analysis;

$$/\sqrt{\left(\frac{}{} \quad \left(\qquad\right)\right)}$$

$T_{m-1,0}$ = the (spectral) mean energy wave period (s), equal to m_{-1}/m_0 α =

Slope angle of structure (^{o})

Δ = Relative mass density of armour = (ρr / ρw) - 1

Dn50 = Nominal rock diameter equivalent to that of a cube (m)

P = Notional permeability factor; the value of this parameter should be: 0.1 ≤ p ≤ 0.6 (CIRIA 1991)

S = Damaged level = A / D2 n50A = Erosion area in a cross-section (m^2). C_{pl}

= 8.4 (with a standard deviation of σ = 0.7, from (CIRIA 1991))

C_s = 1.3 (with a standard deviation of σ = 0.15, from (CIRIA 1991))

g = Acceleration due to gravity (m/s^2).

The transition from plunging to surging waves is derived from the structure slope (not from the slope of the foreshore), and can be calculated using a critical value of the surf similarity parameter ξ_{cr}:

$$[— \qquad \sqrt{\qquad}\,]$$

Depending on the slope angle and permeability, this transition lies between ξ_{cr} = 2.5 to 3.5. When the value of surf similarity parameter is greater than ξ_{cr} , the formula for surging waves is used. For slope angles more gentle than 1:4 (cotα≥ 4), the transition from plunging to surging does not exist and for these slopes, the formula for plunging waves are used irrespective of whether the surf similarity parameter is smaller or larger than the critical value, ξ_{cr}.

The notional permeability factor P should lie between 0.1 for a relatively impermeable core to 0.6 for a virtually homogeneous rock structure. Where data are not available for a detailed assessment, P may be taken as 0.3 for rock armoured structure, unless an open core is to be provided. If in doubt, it is recommended that the permeability be underestimated rather than over-estimated.

The damage level S is the number of cubic stones with a side of D_{n50} being eroded around the water level with a width of one D_{n50}. The limits of S depend mainly on the slope of the structure. For a two-diameter thick armour layer, the lower and upper damage levels have been assumed to be the values shown in Table A1. The start of damage of S = 2 to 3 is the same as that used by Hudson, which is roughly equivalent to 5% damage. Failure is defined as exposure of the filter layer.

The formulae can be used when the number of waves N, or storm duration, is in the range of 1000 to 7000. For N greater than 7000, the damage tends to be overestimated. Unless data are available for more detailed assessment, values of N from 3000 to 5000 may be used for preliminary design purpose (BSI, 1991). The slope of the armour structure, cotα , should lie between 1.5 and 6. The wave steepness s_m should be within the range of 0.005 and 0.06. Waves become unstable when the steepness is greater than 0.06.

Table A1 Damage Levels for Two-Diameter Thick Rock Slopes

Slope of Structure A	Damage Level S at Start of Damage	Damage Level S at Failure
1:1.5	2	8
1:2.0	2	8
1:3.0	2	12
1:4.0	3	17
1:6.0	3	17

Table A2. Range of validity of parameters in Van der Meer formulae for shallow water conditions (CIRIA 1991)

Parameter	Symbol	Range
Slope angle	$\tan\alpha$	1:4–1:2
Number of waves	N	< 3000
Fictitious wave steepness based on T_m	s_{om}	0.01–0.06
Surf similarity parameter using T_m	ξ_m	1–5
Surf similarity parameter using $T_{m-1,0}$	$\xi_{s-1,0}$	1.3–6.5
Wave height ratio	$H_{2\%}/H_s$	1.2–1.4
Deep-water wave height over water depth at toe	H_{so}/h	0.25–1.5
Armourstone gradation	D_{n85}/D_{n15}	1.4–2.0
Core material – armour ratio	$D_{n50\text{-}core}/D_{n50}$	0–0.3
Stability number	$H_s/(\Delta D_{n50})$	0.5–4.5
Damage level parameter	S_d	< 30

Appendix 2 Design of Reef Section - Submerged

Program Developed by Dr Ir P.V.Chandramohan

*

Parameter	Value
High Tide level	1.0 m
Storm surge	1.0 m
Higher SWL	2.0 m
Low Tide level	0.0 m
Lower SWL	1.0 m
Level of crest of reef	0.0 m
Level of bed at the reef	4.5 m
Period of wave	8.0 s
Design wave height at high SWL	5.10 m
Design wave height at Low SWL	4.35 m
Wave Length at High SWL of the reef	59.2 m
Wave Length at Low SWL of the reef	55.0 m
Density of Rock	2.65 T/m^3
Damage factor	2.00
Side slope of reef as tangent of the angle	2.50
Layer coefficient of main armour	1.15
Porosity of main armour as ratio	0.37
Diffraction coefficient in the lee	0.50
Number of layers of the main armour	2
Number of layers in toe	1
Water depth at High SWL	6.50 m
Water depth at Low SWL	5.50 m
Height of the reef	4.50 m

A. Computations for High SWL case

Parameter	**Value**
Structure height to water depth ratio at HWL	0.692
Sp	0.086
Equivalent cube length at High water	0.815 m
Weight of unit - High water	1.435 t
Details of the section on the sea side	
Wave height at the structure	5.10 m
Weight of the Units in outer layer	1.44 Tons
Thickness of outer	1.88 m
Packing density of outer layer	2.18/m^2 =218.1/ 100m^2
Weight of the unit in under layer 1	0.144 Tons
Thickness of under layer 1	0.87 m
Packing density of units in under layer 1	10.13/m^2 = 1013.1/100m^2
Weight of the stone in under layer-2	0.007 Tons
Thickness of under layer-2	0.32 m
Packing density of units in under layer 2	74.72/m^2 = 7472.3/100m^2
Weight of the stone in toe	0.144 Tons
Thickness of toe protection	44 m
Packing density of blocks in toe protection	5.07/m^2 = 506.6/100m^2
Weight of core material	0.4 kg
Details of the section on the lee side	

Diffracted wave height in the lee	2.55 m
Weight of the armour blocks	0.36 Tons
Thickness of main armour layer	1.18 m
Packing density of main armour blocks	$5.5/m^2$ = $550.1/100m^2$
Weight of the block in under layer-1	0.036 Tons
Thickness of under layer-1	0.55 m
Packing density of blocks in under layer-1	$26./m^2$ = $2555.3/100m^2$
Weight of the stone in under layer-2	0.002 Tons
Thickness of under layer-2	0.20 m
Packing density of blocks in under layer-2	$188./m^2$ = $18847./100m^2$
Weight of the stone in toe	0.036 Tons
Thickness of toe protection	0.27 m
Packing density of blocks in toe protection	$12.78/m^2$ = $1277.7/100m^2$
Weight of core material	0. kg

B. Computations for Low SWL case

Parameter	**Value**
Structure height to water depth ratio at LTL	0.818
SpL	0.079
Equivalent cube length at Low water	0.831 m
Weight of unit - Low water	1.523 t
Details of the section on the sea side	
Wave height at the structure	4.35 m
Weight of the Units in outer layer	1.52 Tons
Thickness of outer	1.91 m
Packing density of outer layer	$2.10/m^2$ =$209.7/100m^2$

Weight of the unit in under layer-1	0.152 Tons
Thickness of under layer-1	0.89 m
Packing density of units in under layer-1	9.74/m^2 =973.9/100m^2
Weight of the stone in under layer-2	0.008 Tons
Thickness of under layer-2	0.33 m
Packing density of units in under layer-2	71.83/m2 = 7182.9/100m^2
Weight of the stone in toe	0.152 Tons
Thickness of toe protection	0.44 m
Packing density of blocks in toe protection	4.87/m^2 = 486.9/100m^2
Weight of core material	0.4 kg
Details of the section on the lee side	
Diffracted wave height in the lee	2.17 m
Weight of the armour blocks	0.38 Tons
Thickness of main armour layer	1.21 m
Packing density of main armour blocks	5.5/m2 = 550.1/ 100m2
Weight of the block in under layer-1	0.038 Tons
Thickness of under layer-1	0.56 m
Packing density of blocks in under layer-1	25./m2 = 2456.3/100m2
Weight of the stone in under layer-2	0.002 Tons
Thickness of under layer-2	0.21 m
Packing density of blocks in under layer-2	181./m2 = 18116./100m2
Weight of the stone in toe	0.038 Tons
Thickness of toe protection	0.28 m
Packing density of blocks in toe protection	12.28/m2 = 1228.2/100m2
Weight of core material	0. kg

Parameter	Value
High Tide level	1.0
Storm surge	1.0
Higher SWL	2.0
Low Tide level	0.0
Lower SWL	1.0
Level of crest of the reef	0.0
Level of bed at the reef	1.5
Period of wave	8.0 s
Design wave height at high SWL	2.81 m
Design wave height at Low SWL	2.02 m
Wave Length at High SWL	45.0 m
Wave Length at Low SWL	38.4 m
Density of Rock	2.65 T/m^3
Damage factor	2.00
Side slope of breakwater as tangent of the angle	2.50
Layer coefficient of main armour	1.15
Porosity of main armour as ratio	0.37
Diffraction coefficient in the lee	0.50
Number of layers of the main armour	2
Number of layers in toe	1
Water depth at High SWL	3.50 m
Water depth at Low SWL	2.50 m
Height of BW	1.50 m

Computations for High SWL case	
Structure height to water depth ratio at HWL	0.429
Sp	0.062
Equivalent cube length at High water	0.357 m
Weight of unit - High water	0.121 t
Details of the section on the sea side	
Wave height at the structure	2.81 m
Weight of the Units in outer layer	0.12 Tons
Thickness of outer	0.82 m
Packing density of outer layer	11.36/m^2 = 1136.4/ 100m^2
Weight of the unit in under layer-1	0.012 Tons
Thickness of under layer-1	0.38 m
Packing density of units in under layer-1	52.79/m^2 = 5278.8/100m^2
Weight of the stone in under layer-2	0.001 Tons
Thickness of under layer-2	0.14 m
Packing density of units in under layer-2	389.33/m^2 = 38933.1/100m^2
Weight of the stone in toe	0.012 Tons
Thickness of toe protection	0.19 m
Packing density of blocks in toe protection	26.39/m^2 = 2639.4/100m^2
Weight of core material	0 kg
Details of the section on the lee side	
Diffracted wave height in the lee	1.40 m
Weight of the armour blocks	0.03 Tons
Thickness of main armour layer	0.52 m
Packing density of main armour blocks	28.7/m^2 = 2866.2/ 100m^2
Weight of the block in under layer-1	0.003 Tons
Thickness of under layer-1	0.24 m
Packing density of blocks in under layer-1	133./m^2 = ******/100m2
Weight of the stone in under layer-2	000 Tons
Thickness of under layer-2	0.09 m
Packing density of blocks in under layer-2	982./m^2 = 98196./100m^2
Weight of the stone in toe	0.003 Tons
Thickness of toe protection	0.12 m
Packing density of blocks in toe protection	66.57/m^2 = 6657.0/100m^2
Weight of core material	0. kg

Computations for Low SWL case	
Structure height to water depth ratio at LTL	0.600
SpL	0.053
Equivalent cube length at Low water	0.340 m
Weight of unit - Low water	0.104 t
Details of the section on the sea side	
Wave height at the structure	2.02 m
Weight of the Units in outer layer	0.10 Tons
Thickness of outer	0.78 m
Packing density of outer layer	12.55/m^2 = 1254.9/ 100m^2
Weight of the unit in under layer-1	0.010 Tons
Thickness of under layer-1	0.36 m
Packing density of units in under layer-1	58.29/m^2 = 5829.1 /100m^2
Weight of the stone in under layer-2	0.001 Tons
Thickness of under layer-2	0.13 m
Packing density of units in under layer-2	429.92/m^2 = 42991.8/100m^2
Weight of the stone in toe	0.010 Tons
Thickness of toe protection	0.18 m
Packing density of blocks in toe protection	29.15/m^2 = 2914.5/100m^2
Weight of core material	0 kg
Details of the section on the lee side	
Diffracted wave height in the lee	1.01 m
Weight of the armour blocks	0.03 Tons
Thickness of main armour layer	0.49 m
Packing density of main armour blocks	28.7/m^2 = 2866.2/ 100m^2
Weight of the block in under layer-1	0.003 Tons
Thickness of under layer-1	0.23 m
Packing density of blocks in under layer-1	147./m^2 = ******/100m2
Weight of the stone in under layer-2	.000 Tons
Thickness of under layer-2	.08 m
Packing density of blocks in under layer-2	1084./m^2 = 108433./100m^2
Weight of the stone in toe	0.003 Tons
Thickness of toe protection	0.11 m
Packing density of blocks in toe protection	73.51/m^2 = 7350.9/100m^2
Weight of core material	0. kg

Appendix 3: Concrete Caisson Design Parameters A

Wedge

1.Full sand					
Depth below Hip		1	m	Slab thickness	0.4
Concrete density		25	kN/m3		
Sand density		18	kN/m3		
Water density		10.1	kN/m3		
Width of caisson		50	m		
Water density		1.025	kN/m3		
Force					
Weight of caisson			Force	Moment	
Concrete	20	kN/m2	1000	25000	kNm
Sand fill	18	kN/m2	900	22500	kNm
Triangular fill	18	kN/m2	900	22500	kNm
Weight of water on top	20.5	kN/m2	1025	25625	
Total about toe			**2800**	**70000**	kNm
Overturning					
Dynamic uplift	18	kN/m2	900	30000	kNm
Static	23	kN/m2	1153	28828	kNm
Total about toe			2053	58828	
F.S				1.19	
2.Sand filled only for a fraction of the width					
Fraction	0.5		Lever arm	0.75	
Force					
Weight of caisson			Force	Moment	
Concrete	20	kN/m2	1000	25000	kNm
Sand fill	18	kN/m2	450	16875	kNm
Triangular fill	18	kN/m2	450	16875	kNm
Total about toe			**1900**	**58750**	kNm
Overturning					
Dynamic uplift	18	kN/m2	900	30000	kNm
Static	23	kN/m2	1153	28828	kNm
Total about toe			2053	58828	
F.S				1.00	

B Double Turtle Wedge

Double turtle								
Wave subsurface pressure at -2.5	63.88	kPa	Density of concrete		25	kN/m3		
Wave subsurface pressure at 0.0	41.17	kPa	Beam spacing	4	m			
Angle of the surface to horz	5.71	deg	Width of beam	0.5	m	500		
Strength of concrete	40	Mpa	Depth of rib	0.5	m	500		
Strength of steel	500	MPa	Col spacing	4	m			
1 Top Slab - subsurface pressure			Col size	300	mm			
Span of the slab	3.7	m	3700	mm				
Design as two way								
Self weight of slab	5	kPa						
Max BM	30	kNm						
Mu	45	kNm						
Dia of bar	10	mm	Area	78.54				
cover	50	mm						
Full depth of slab	200	mm						
Eff depth	145	mm						
Mu/bd2	2.153	<	5.32	ok				
Mu	46	0.0054						
As	783	mm2						
No of bars/m width	10.0							
Spacing	100	mm c/c						
Shear need not be considered								
Beam								
Two way slab - transfer of load to beam								
Self weight of slab	40	kN						
Wave force	128	kN						
Total	168	kN	41.94	kN/m				
Rib of beam	6.25	kN/m						
Total load	174	kN/m						
BM	278	kNm						
Mu	418	kNm						
Total depth	700	mm						
Dia of bar	16	mm	Area	201.06				
Eff depth	642	mm						
Mu/bd2	2.03	<	5.32	ok				

Mu	428	0.0051					
As	1637	mm2					
No of bars/m width	8						
Spacing	49	mm c/c					
Shear in the beam	348	kN					
Vu	522						
Shear stress	1.63	MPa					
Allowable	0.51	MPa					
Shear taken by concrete	165	kN					
Shear to be taken by steel	357	kN					
Provide stirrups	10	mm dia	2	legged	area	157	mm2
Spacing	102	mm c/c					
Column							
Load on the column	1392	kN					
Pu	2088	kN					
Dia of bar	12	mm	Area	113.1	mm2		
d'	56	mm					
d'/D	0.19						
Pu/fckbD	0.580						
Mu	0						
Chart 50 SP 16							
p/fck	0.04	p=	1.6				
As	1440	mm2					
No of rods	12.73						
Provide 4Nos on each side							
Bottom slab - Uplift pressure							
Caisson will be floating. Full draught will be 5m. But during floating, the draught will be 4m.							
This has been already taken care of in the subsurface pressure design.							
The pressure taken is	63.88	kPa					
Pressure computations have been made by a program.							

Appendix 4: South Reef Steel Caisson Design Parameters

Program Developed by Dr Ir P.V.Chandramohan

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Parameter	Value
Depth at the location	4.00 m
Highest Astronomical tide	+ 1.00
Storm surge	1.00 m
Period of the wave	8.00 seconds
Slope of the sea bed	0.0100
Refraction coefficient	1.0
Height of rubble bed	0.0 m
Thickness of outer layer	0.0 m
Top level of caisson on lee side	+ 1.00 m
Top level of caisson on sea side	+ 2 m
Significant wave height	4.73 m
Angle between crest and structure	0.0 degrees
Width of caisson	18.00 m
Bottom level of rubble bed	-4.00 m
Length of the caisson	25.00 m
Projection of footing beyond caisson side	3.200 m
Thickness of bottom slab	0.020 m
Thickness of top slab	0.020 m
Thickness of wall of caisson	0.020 m
Mass density of sand fill	1.800
Friction coefficient between caisson and rubble	0.50
Mass density of steel	7.650

Width of berm	0.0 m
Height of water ballast during floating	
Number of cells across width of caisson	6 nos
Number of cells along length	8 Nos
Design wave height	4.730 m
Wave length at the location	57.50 m
Option for pressure for computation	5