Figure 3.5: Applications of sandbags for coastal protection and reclamation works

Such sandbags are available in various sizes and constructed of various types of materials. Sand bags most commonly used for coastal protection works in South Africa are made from a multi-layered needle-punched geotextile.

Sandbags are most suited for applications where wave energy is low, such as in lagoons and lakes, or where they are exposed to wave action infrequently or for short periods, such as for emergency measures or applications above the typical high water mark. They are generally not used on rocky areas due to the risk of rocks abrading or puncturing the bags.

4 Wave Modelling

A wave transformation modelling study was undertaken in order to derive the nearshore wave conditions. The modelling process and results are described in this section.

4.1 Model Background and Setup

4.1.1 Description of wave refraction model

The third-generation wave generation and refraction model SWAN (Simulating WAves Nearshore (Booij, et al, 1999)) was applied. SWAN was run within the DELFT3D-WAVE environment (Deltares, 2013), which provides a convenient interface for pre- and post-processing of the results.

The SWAN model is based on the discrete spectral action balance equation and is fully spectral (in all directions and frequency), implying that short-crested random wave fields propagating simultaneously from widely different sources can be accommodated, e.g. a swell with superimposed wind sea. SWAN computes the evolution of random, short-crested waves in coastal regions with deep, intermediate and shallow water.

The SWAN model accounts for refractive propagation due to depth changes. The model also represents explicitly the processes of wave generation by wind, dissipation by whitecapping, bottom friction and depth-limited wave breaking and non-linear wave-wave interactions (quadruplets and triads) with state-of-the-art formulations.

In the present application, the processes simulated were: wave refraction, shoaling, bottom friction, and breaking. Local wind-wave generation within the area modelled was not included.

4.1.2 Model set-up

The following particulars apply to the model setup.

- The coordinate system used is Lo17 Cape Datum. This is the coordinate system used on the mine.
- Wave directions refer to the directions from which the waves come, measured clockwise from north.
- All wave heights are the significant wave height (Hs) unless indicated otherwise, while periods are the spectral peak period (Tp), unless indicated otherwise.
- All depths are to Mean Sea Level (MSL).
- Bathymetry obtained from chart BA2078 & parts of SAN115 (courtesy of the SANHO), and recent the survey by CSIR in May 2015.

The model domain is shown in Figure 4.1. A system of nested bathymetric and computational grids was used to allow increased resolution, and thus computational accuracy, in the area of interest and as water depths decreased towards the shore. Grid cell sizes ranged from 500 x 500 m at the deep-water boundaries to 20 x 20 m at the project sites. Detail of the inner grid and bathymetry is shown in Figure 4.2 and Figure 4.3 for Rooiwal Bay and Site 68/69 respectively.

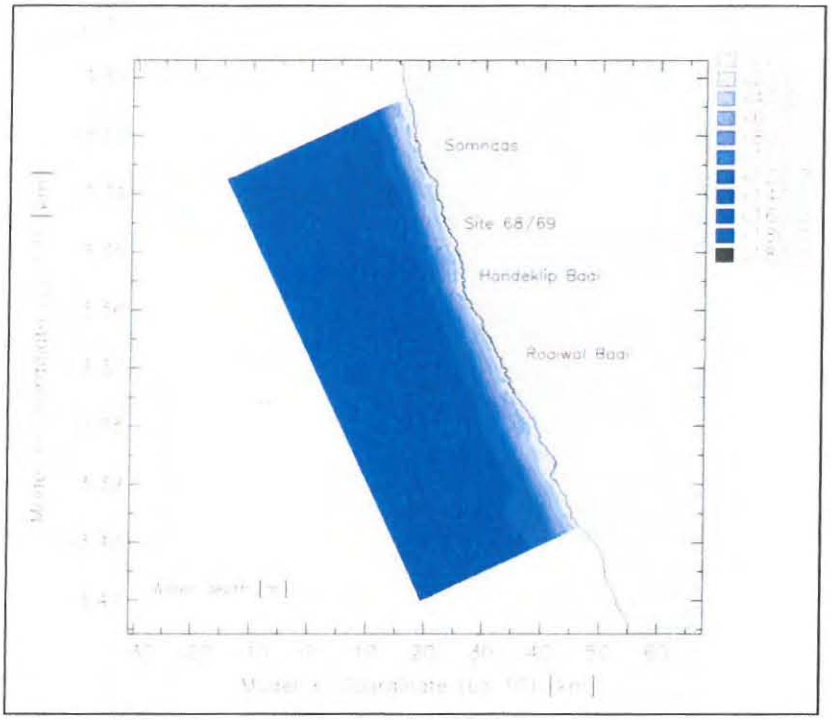

Figure 4.1: Layout of the computational domain and bathymetry

The coarser (larger) grid extends approximately 30 km offshore to a water depth of 150 m and extends 80 km along the coast, from 40 km north of Hondeklip Baai, to 40 km south of it.

The fine grid for Rooiwal extends 600 m offshore to deeper than 15 m water depth and is 1200 m wide, covering the entire bay.

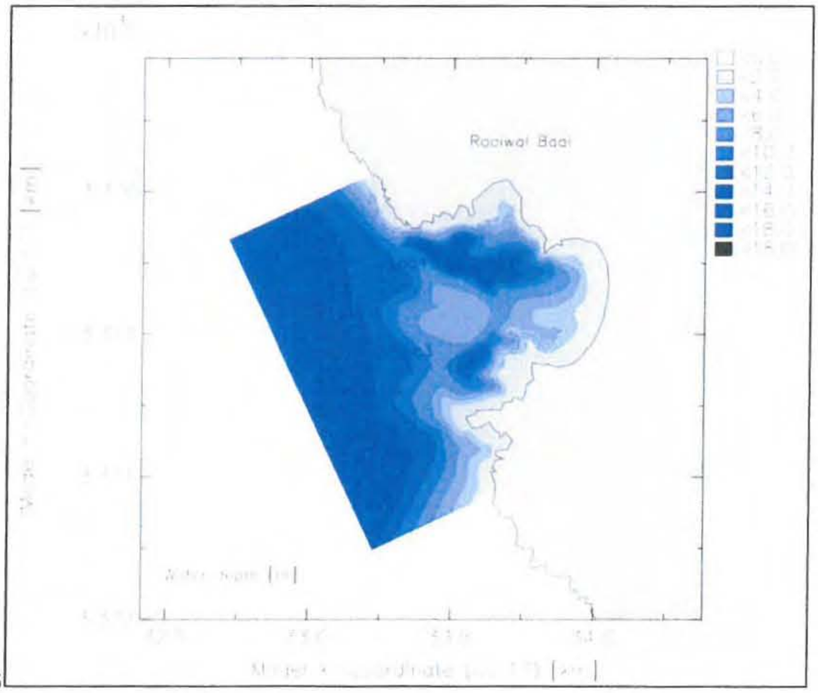

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Figure 4.2: Layout of the fine computational domain and bathymetry at Rooiwal Bay

The fine grid at Site 68/69 extends 1400 m offshore to a water depth of approximately 15 m and is 2700 m wide.

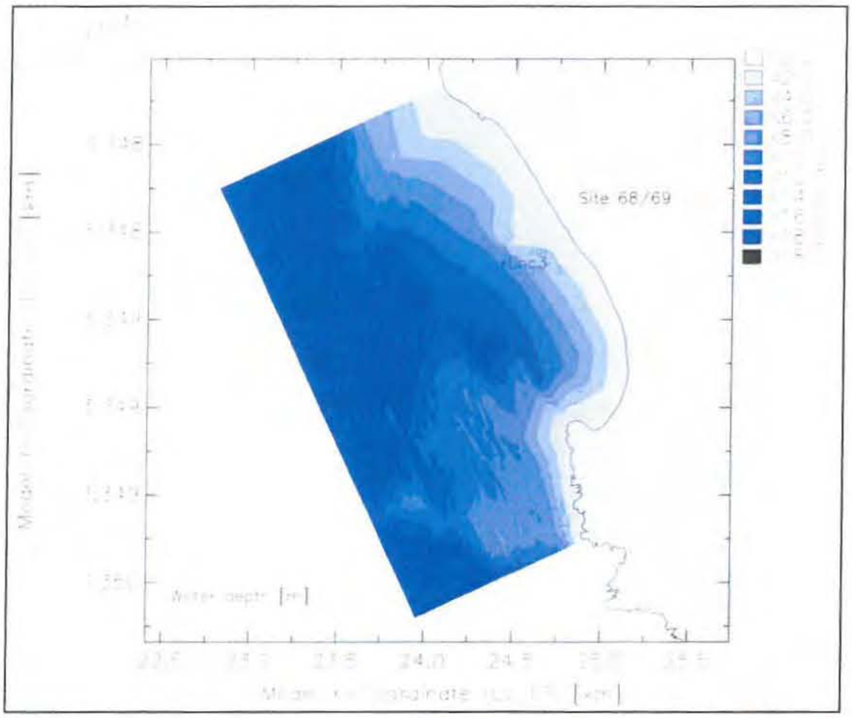

Figure 4.3: Layout of the fine computational domain and bathymetry Site 68/69

4.1.3 Wave Transformation Matrix

A transformation matrix was developed that would allow any offshore wave condition to be transformed to the nearshore. The matrix was constructed by transforming a set of offshore conditions of varying height, period and direction, to nearshore. The discretised wave conditions are presented in Table 4.1. The various combinations of wave direction, period and height totalled 280 different boundary conditions.

This method allowed more than 99% of the offshore NCEP dataset to be transformed to nearshore, including all waves larger than 5 m.

Table 4.1: Conditions used to develop combinations of offshore boundary conditions

Wave directions [°N]	Wave periods [s]	Wave heights [m]
180.0	6.0	0.5
202.5	8.0	2.0
225.0	10.0	3.5
247.5	12.0	5.0
270.0	14.0	6.5

	16.0	8.0
	18.0	9.5
	20.0	

4.2 Model Calibration

No local wave measurements are available for calibration of the SWAN model. The NCEP data, along with the SWAN model is, however, sufficient to provide an initial estimate of the expected wave conditions at the area of interest. Default parameters were used within the SWAN model. These have proven to be reliable for other studies along the west coast and elsewhere.

It would be prudent to measure actual wave conditions at the sites during detailed design stage, in order to verify the model results.

4.3 Results

Figure 4.4 to Figure 4.9 present the offshore and nearshore wave conditions for both Rooiwal Bay and Site 68/69. Offshore south-westerly waves with height between 1.0 m and 3.5 m account for over 50% of the recorded wave climate (with wave periods of between 10 s to 14 s). As illustrated in Figure 4.4, the waves from this direction propagate straight towards the coastline with minimal refraction. Bottom friction is the only major mechanism contributing to reducing wave energy closer to shore.

The wave output at numerous locations were extracted (illustrated in Figure 4.2 at locations 2, 3 and 4 and in Figure 4.3 at locations 1, 2 and 3) and used to determine the return periods, used later in this report.

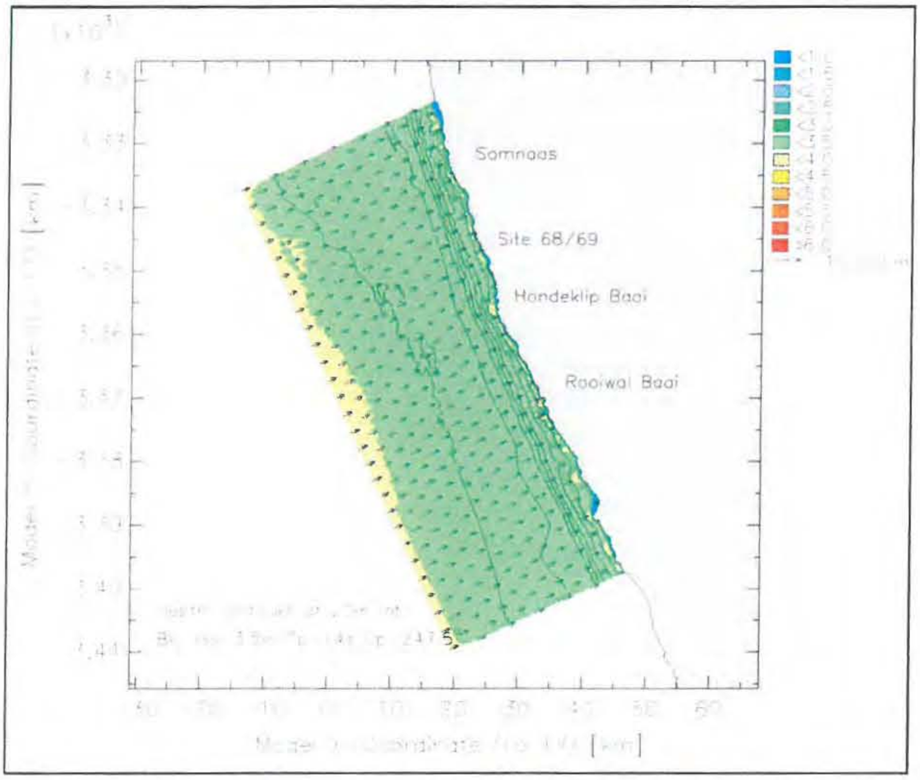

Figure 4.4: Wave height and direction transformation on the offshore grid

Figure 4.5 shows how nearshore features significantly affect the wave field as it enters the bay. Sudden changes in the local bathymetry will affect the wave direction as the waves refract to shallower water. With the sudden changes and irregular bathymetry present in Rooiwal Baai, the results obtained near and downstream of these bathymetric features must be interpreted with caution.

The bay at Site 68/69 (Figure 4.6) contains numerous rocky outcrops and bathymetric features, with steep and curved contours. This is synonymous with reefs and patches of smoother sandy seabed. The irregular sea bottom induces wave shoaling. However, the direction of the wave propagation is not affected noticeably by the sudden changes in the bathymetry. The refraction of the waves towards shallower water, as indicated by arrows in the figures, is as anticipated.

Figure 4.7 illustrates the wave height and direction transformation of a typical storm wave condition, with height of 5 m, and period of 14 s and south-westerly direction respectively. This wave height is exceeded by only 1.5% of the NCEP data. Wave height and direction changes become noticeable landward of the 60 m depth contour.

The modelled conditions at Rooiwal Bay and Site 68/69 for this offshore condition are shown in Figure 4.8 and Figure 4.9 respectively.

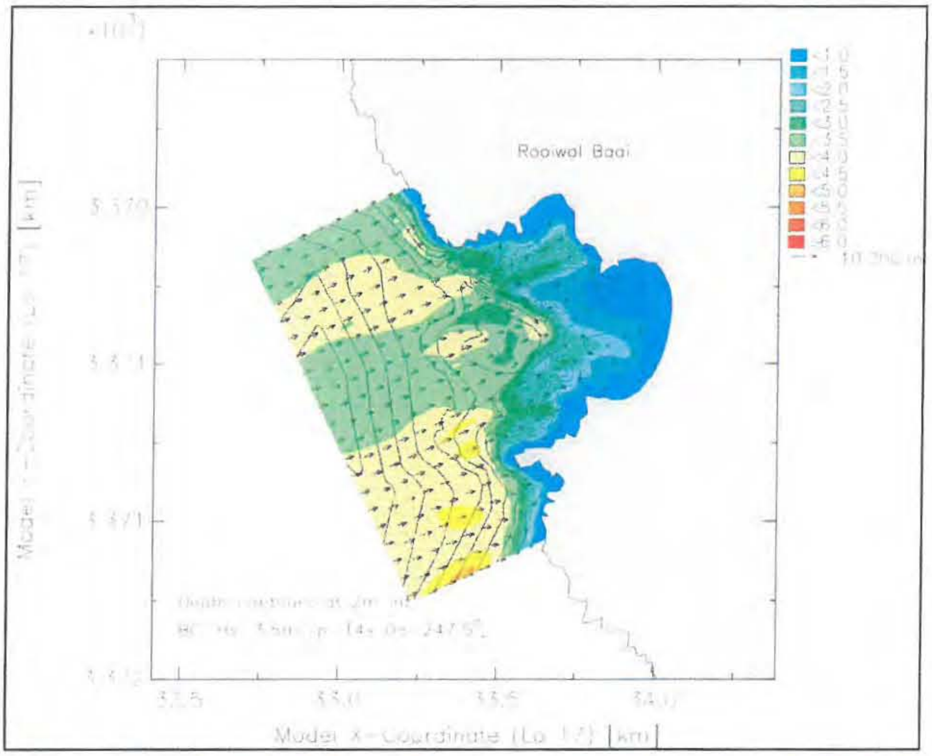

Figure 4.5: Wave height and direction transformation on the nearshore grid at Rooiwal Bay

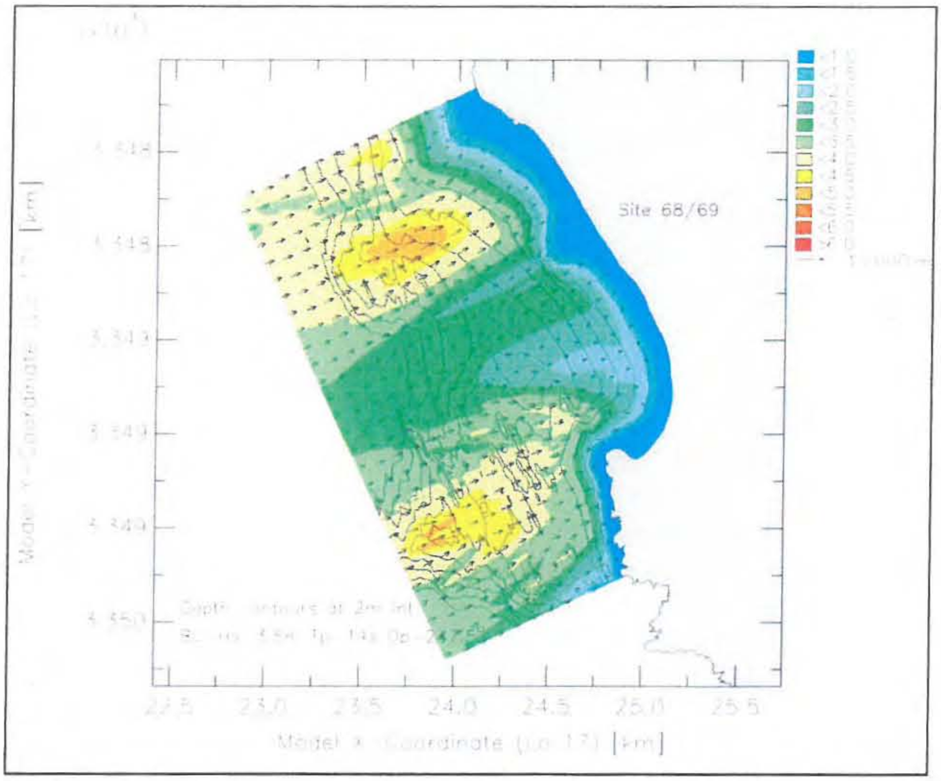

Figure 4.6: Wave height and direction transformation on the nearshore grid at Site 68/69

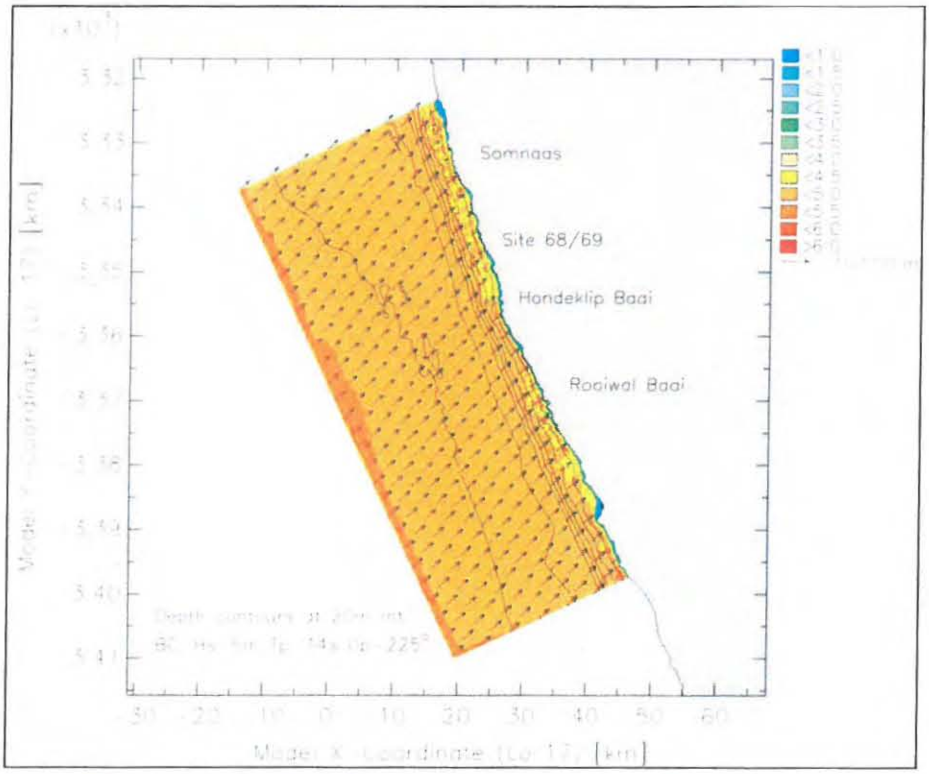

Figure 4.7: Wave height and direction transformation on the offshore grid

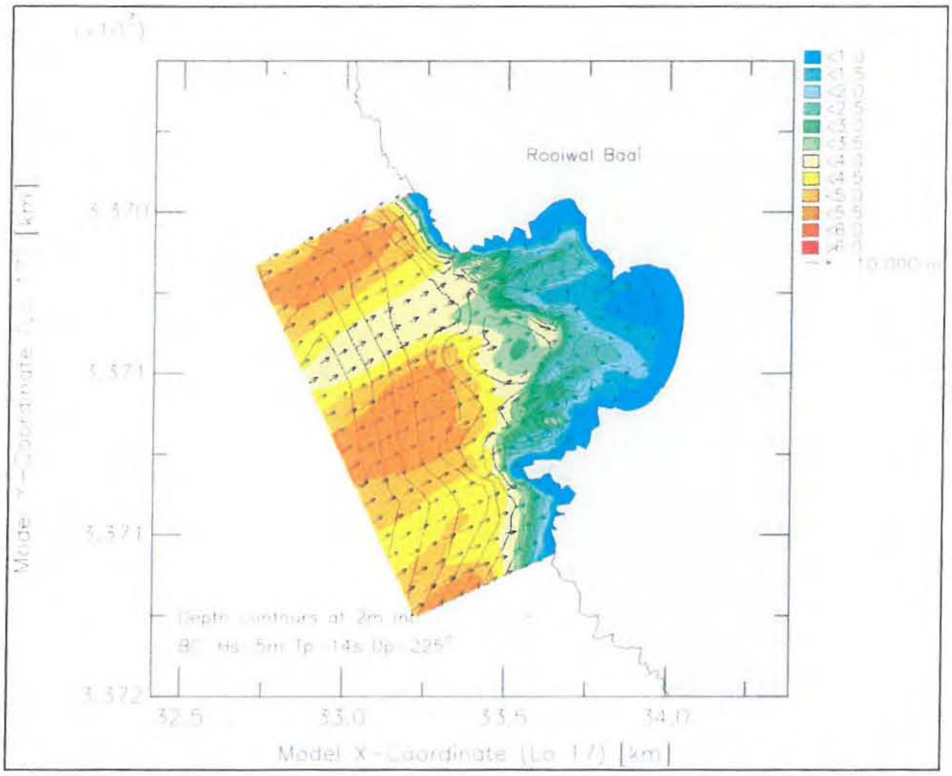

Figure 4.8: Wave height and direction transformation on the nearshore grid at Rooiwal Baai

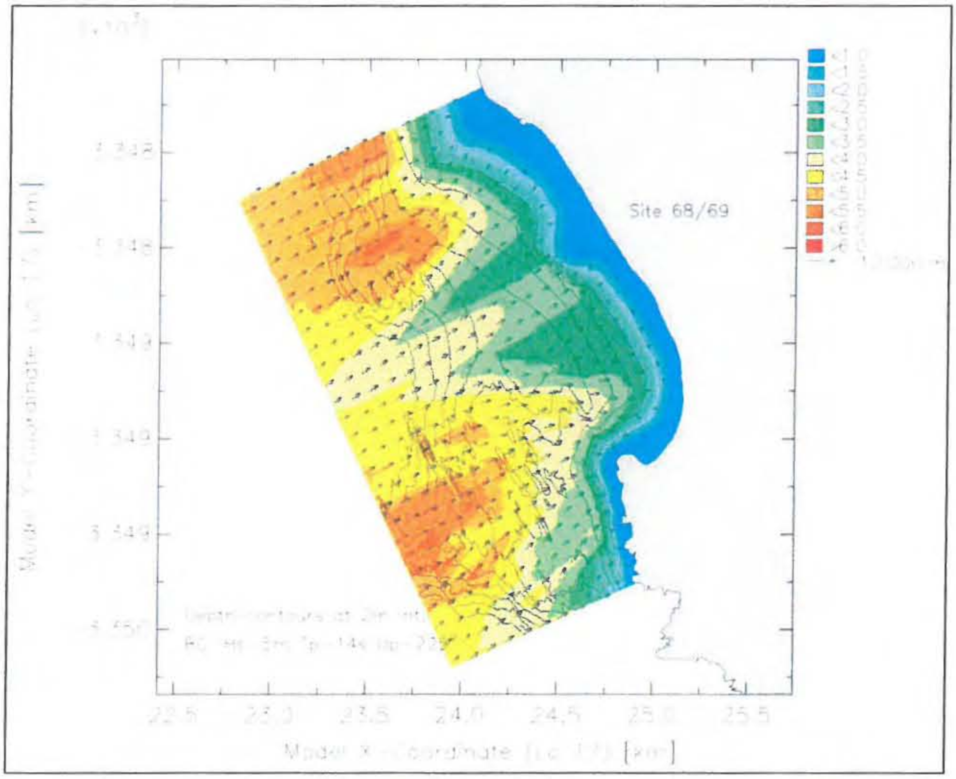

Figure 4.9: Wave height and direction transformation on the nearshore grid at Site 68/69

5 Beach Mining Solutions for Rooiwal Bay

Two areas have been identified for mining at Rooiwal Bay:

- The small northern channel area; and
- The greater bay area.

Approaches for mining each of these areas are described in the following sections.

5.1 Berm across Northern Channel

The northern channel is a narrow channel that enters the bay on its northern shore of the bay. The inland part of the channel has been mined previously. Further mining would involve extending the mining area southward.

5.1.1 Approach

A protective berm will be required to close off the channel from the sea. A rock berm would be required, as sand-size material would erode easily. A method to construct the berm would be to end-tip rock from the western side towards the east to close the channel.

It is understood that the mining would be completed in a short time (<1 year). The design life of the berm could thus also be short. Equipment and materials would be on hand to repair wave damage. The 1:1 year return period wave height is estimated to be 3 m at the position of the northern channel. This estimate takes into account that wave heights are limited by breaking that occurs seaward of this point, and by the local water depths.

A requirement is to minimise the width of the berm. This will prevent the landward toe of the berm encroaching on the mining area, and prevent the seaward toe extending into deepwater which would require larger volumes of rock.

5.1.2 Layout

The channel is located inside Rooiwal Bay where wave energy is reduced. However, the coastline east of the channel is exposed to wave energy entering the bay over the deep water in the north of the bay. A berm would be exposed to an increasing amount of wave energy the further into the bay (southward) it is built.

A layout of the berm is shown in Figure 5.1. The berm is oriented approximately east-west across the junction of the channel and the bay proper. The length of berm required is estimated to be 140 m.

Figure 5.1: Northern channel closure berm concept

The coastline at the berm location is orientated at an angle to the approaching waves. Waves would therefore attack the berm at an angle. This would reduce the stability of the stones in the armour layer – the stones would tend to move eastward with the incoming wave direction. Large rock sizes would be required for stability. If smaller rock is used then frequent maintenance (replenishment) would be needed to replace stones that are washed off the slope.

5.1.3 Conceptual Berm Section

The parameters for a conceptual design of the berm cross-section are given in the table below.

Table 5.1: Berm cross-section

Parameter	Value	Assumptions/Notes
Crest elevation	+6.5 m MSL	Overtopping should not occur.
Crest width	6 m	Vehicle access in only one direction is considered adequate, as the berm length is short (140 m).
Seaward slope	1v: 3h	Steeper slopes will require larger armour sizes
Armour rock size	Median: 2.2 tonnes Range: 1.5 to 4.5 tonnes	Two layers of armour rock would be required. Stones larger than about 3 tonnes become difficult to transport, handle and place without a crane.
Core rock size	250 kg 25 kg	An underlayer consisting of 250 kg stones would be required under the armour rock. The mass of the underlayer should be approximately 1/10th of the armour layer. The mass of the core material should be approximately 1/10th mass of the underlayer, i.e. 25 kg. Finer material can be used on the landward side of the core in order to reduce permeability.

Wave runup on the berm was estimated using the Eurotop method (Eurotop, 2007). During the design wave condition (1:1yr recurrence wave height of 3m) and design water level (+1.4m), the 2-percent exceedance runup level is calculated to be +6.0m MSL. A crest elevation of +6.5m is therefore considered appropriate.

The total rock volume would be approximately 21 000 m^3 (Armour = 9 500 m^3, core 11 500 m^3).

The core and underlayer material are of a size suitable for end-tipping from a haul truck. The armour rock would need to be placed into final position using a large excavator.

The berm would be permeable. The permeability could be reduced by including a vertical layer of fine material on the landward side of the core. This layer should extend upwards to at least the elevation of MHWS.

Removal of the berm for rehabilitation purposes would require removal of part of the armour layer and reducing the crest level. This would allow wave action to overtop the structure and erode the smaller underlayer and core material for eventual redistribution on the seabed..

5.1.4 Cost Estimate

Costs are highly dependent on several factors, amongst them:

- Is suitable rock available, or does it need to be quarried. Quarry establishment costs can be high;
- Haulage distances;
- Plant required for placing the rock – trucks, excavators, crane for large rocks;
- Quantities required – economies of scale can apply;
- Preliminary and general items associated with the construction.

A lower limit for constructing the berm is estimated to be R5 million, excluding P&G items. This is based on short haulage distance to a non-commercial quarry.

An upper limit is estimated to be R12 million, excluding P&G items. This is based on rock sourced from a commercial quarry and hauled distance of approximately 20km.

5.2 Closure of Entire Bay with Rock Berm

5.2.1 Approach

Extensive potential additional mining area exists within the greater Rooiwal Bay. The diamond grades are not known at present. Mining this area with conventional open-cast methods would require closing-off the entire bay with a berm. Such a berm option is investigated here.

The wave transformation modelling (Section 4) was used to provide the basic design wave conditions at the location of the berm. Possible berm options were considered and their conceptual design is described in the next sections.

5.2.2 Layout

A proposed berm layout is shown below. The alignment is such that the berm lies over shallower parts of the seabed, thereby limiting the volume of material required. The berm crosses the reef in the centre of the bay (depth of -4 m). It passes seaward of the channel (depths >-15 m) in the northern part of the bay. The resulting alignment is slightly oblique to the incoming wave direction. The wave height and direction distribution at a point seaward of the berm are shown in the wave rose in Figure 5.2.

The length of the berm is approximately 700 m.

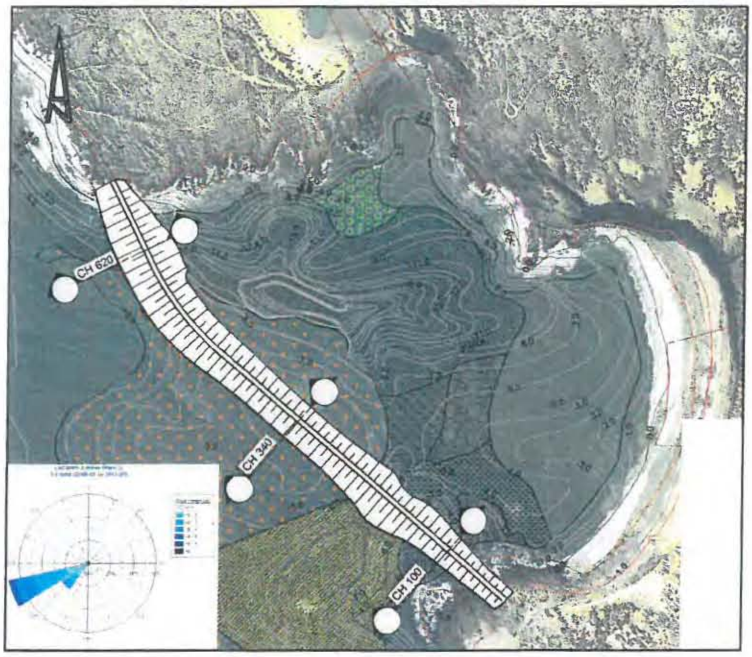

Figure 5.2: Berm extending across Rooiwal Bay. Inset shows wave height and direction seaward of berm

The wave refraction modelling (Section 0) was used to transform the offshore wave conditions to a point seaward of the berm, at approximately 10m water depth. Statistical analysis of this nearshore data yielded the following return period wave heights:

Table 5.2: Rooiwal nearshore return interval wave heights

Return period (years)	Significant wave height (m)
1	5.9
5	6.6
10	6.9

In selecting the appropriate return period, the following should be taken into consideration:

- Intended design life. Including the construction time, it is likely that the structure will need to be in place for 2 to 5 years. It is effectively a temporary structure.
- Acceptability of damage. A lower design life can used for a structure where damage is acceptable and where damage can repaired easily and rapidly. The client has indicated that ongoing maintenance will be acceptable.
- Consequence of failure. Failure of the structure could lead to loss of life. Considerable expense would also be incurred to restore the site after a failure of the berm

A return period of 10 years is considered suitable, giving a wave height of 6.9 m. A wave period of 16 to 18 seconds would be associated with this wave height. These parameters determine the design of the berm section – slope, armour size.

5.2.3 Option 1 – Rock Berm

Similar to the berm across the northern channel, an option is to construct the berm across Rooiwal Bay as a conventional multi-layer breakwater using rock armour. This massive berm would be exposed to unbroken waves – the water depths along the toe range between 4 m and 10 m. Estimation of the armour stone sizes required indicates that required masses would vary from 3 tonnes to 25 tonnes for the shallowest and deepest sections, respectively. The same assumptions of a modest slope (1:3) and generous allowance for damage, and resultant frequent maintenance, were applied. Applying a more conventional slope and a stricter damage criteria results in the required mass of the armour stones increasing two-fold.

This berm solution thus presents several problems:

- The large armour stones are difficult (potentially impossible) to produce from a regular quarry;
- The large stones are difficult to handle and transport. They would be too large to handle with a conventional excavator, requiring a crane. This is undesirable due to the increased cost and complexity of construction;
- The water depth at the site is too deep to allow end-tipping and placement of armour rock on the underwater slope without a crane.

Concrete armour units (e.g. dolosse) would traditionally be used in such a case, in order to overcome the requirement for very large armour stones. Concrete armour units achieve part of their stability through inter-locking between units, and not solely their mass, which allows the units to be lighter in comparison to rock. However, concrete units are not considered suitable for the following reasons:

1. Cost of concrete unit manufacture. A specialist concrete casting yard would need to be set-up at the site in order to manufacture the units.
2. Construction complexity. Placing of concrete units below water is a specialist marine construction activity, requiring an experienced marine contractor.
3. A crane would be required to place the units, as they cannot be end-tipped.
4. They are difficult to remove during rehabilitation.

For these reasons an alternative rock berm solution was investigated.

5.2.4 Option 2 – Dynamically Stable Rock Beach

The main draw-back of the previous option is the large mass of the armour stones required. This is a result of the individual stones needing to be static (minimal movement) on the berm slope in order to maintain the structure's stability.

An alternative approach is to allow the rocks to move in response to changing wave and water level conditions. The resulting dynamically stable profile, or rock beach, adjusts to maintain equilibrium with the prevailing wave conditions. This equilibrium concept is illustrated in Figure 5.3 for three different initial slopes. An example of a rock beach is shown in Figure 5.4.

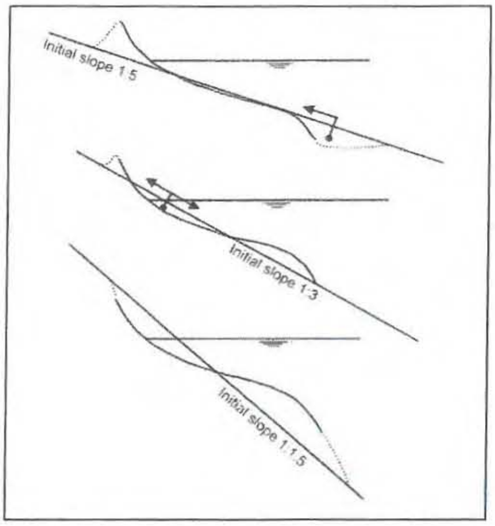

Figure 5.3: Dynamically stable profiles for different initial slopes (CIRIA, 2007)

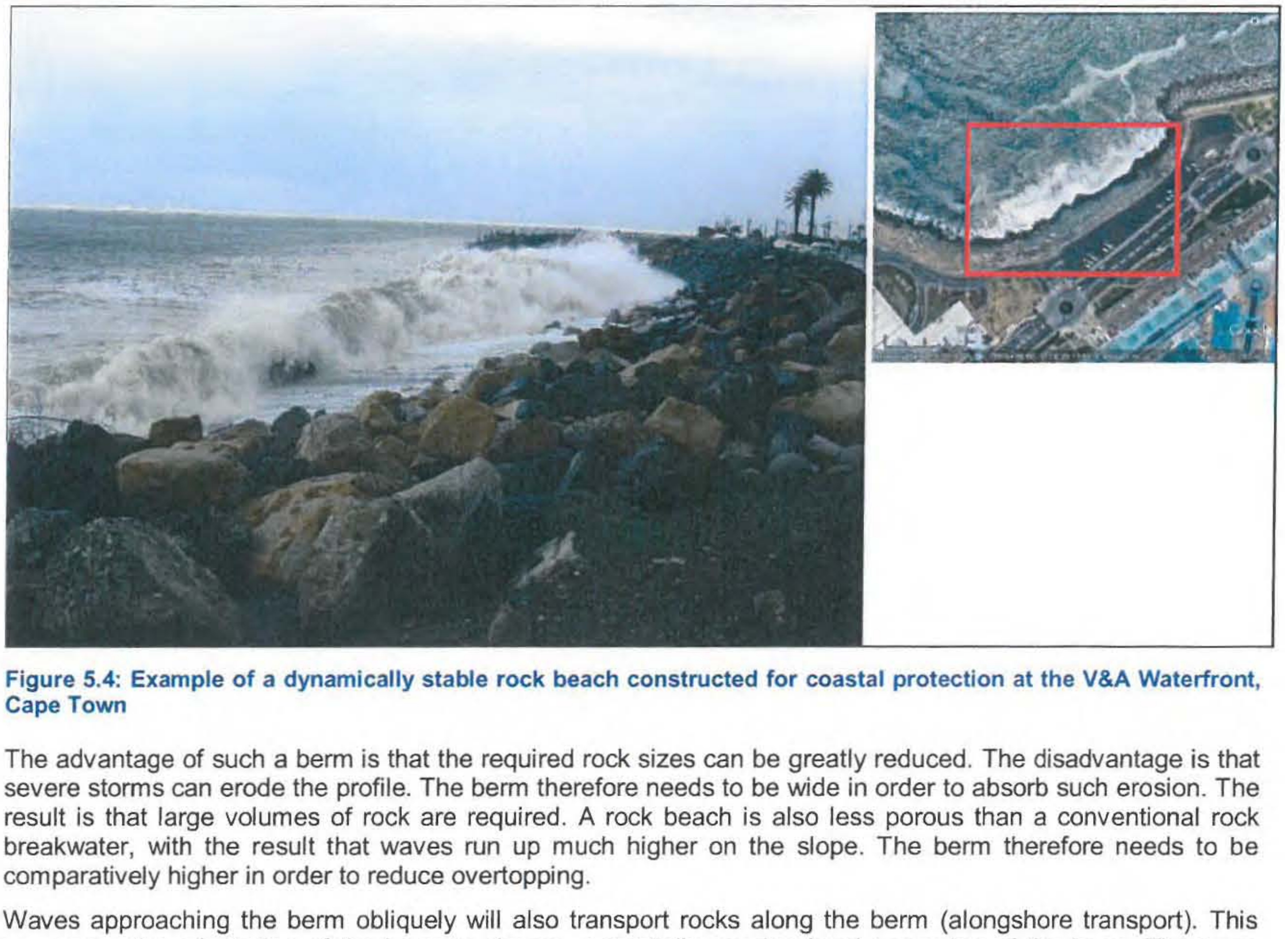

Figure 5.4: Example of a dynamically stable rock beach constructed for coastal protection at the V&A Waterfront, Cape Town

The advantage of such a berm is that the required rock sizes can be greatly reduced. The disadvantage is that severe storms can erode the profile. The berm therefore needs to be wide in order to absorb such erosion. The result is that large volumes of rock are required. A rock beach is also less porous than a conventional rock breakwater, with the result that waves run up much higher on the slope. The berm therefore needs to be comparatively higher in order to reduce overtopping.

Waves approaching the berm obliquely will also transport rocks along the berm (alongshore transport). This can cause the orientation of the berm to change, potentially causing local narrowing of the berm. The berm therefore needs to be aligned perpendicular to the predominant wave direction.

The slope of such a dynamically stable berm is gentler than a conventional rock berm, such as Option 1, and its crest is higher. Large volumes of rock are therefore required to construct the berm, and the footprint of the berm covers a larger area.

Design

Dynamically stable rock berms are not used as frequently for coastal protection as conventional rock berms. Design guidelines are thus more limited. At this concept stage, the method of Van der Meer (1998) was used to estimate the profile shape. The following are relevant:

- Design wave height of 6.9 m, with 16 s period. This determines the height of the berm crest. A berm crest of 14 m was determined for this wave condition. This seems high, but it must be borne in mind that overtopping of the berm should not occur. Crest levels would be lower in those areas where the berm is exposed to smaller waves, such as in the centre of the bay.
- Median rock size (D_{n50}) of 0.3 m, corresponding to a mass (M_{50}) of 70 kg. The size (mass) determines the slope of the profile, with larger material resulting in a steeper slope. The selected size (0.3 m) corresponds to a large cobble/small boulder. Actual rock sizes should vary around this median.

The layout and profile of the berm are shown in Figure 5.5 below. The alignment of the berm was changed relative to that for Option 1 such that the berm is oriented approximately perpendicular to the wave direction. A berm crest width of 10 m was assumed.

It is likely that there would be considerable wastage, or loss, of rock during construction due to erosion by waves. It is possible that the rate at which rock is lost will exceed the rate at which material can be placed, particularly in areas of deep water. Completion of the berm would then not be possible.

Volumes and Cost

The volume of rock required was estimated to be 660 000 m^3. Actual construction volumes would likely be higher than this when losses are taken into account.

A first estimate of the costs of the rock supply and placement, using a lower-end rate and a commercial rate, is between R165 million and R370 million. This amount excludes any P&G items.

Ongoing maintenance, involving replenishment of eroded rock, would be required for the life of the structure.

Rehabilitation

After mining is completed, rehabilitation could be achieved by reducing the berm crest by dozing it landwards and allowing natural erosion to redistribute the rock on the seabed. Alternatively, the material above water could be loaded and hauled to a spoil site on land. It would not be feasible to remove that part of the berm located below water.

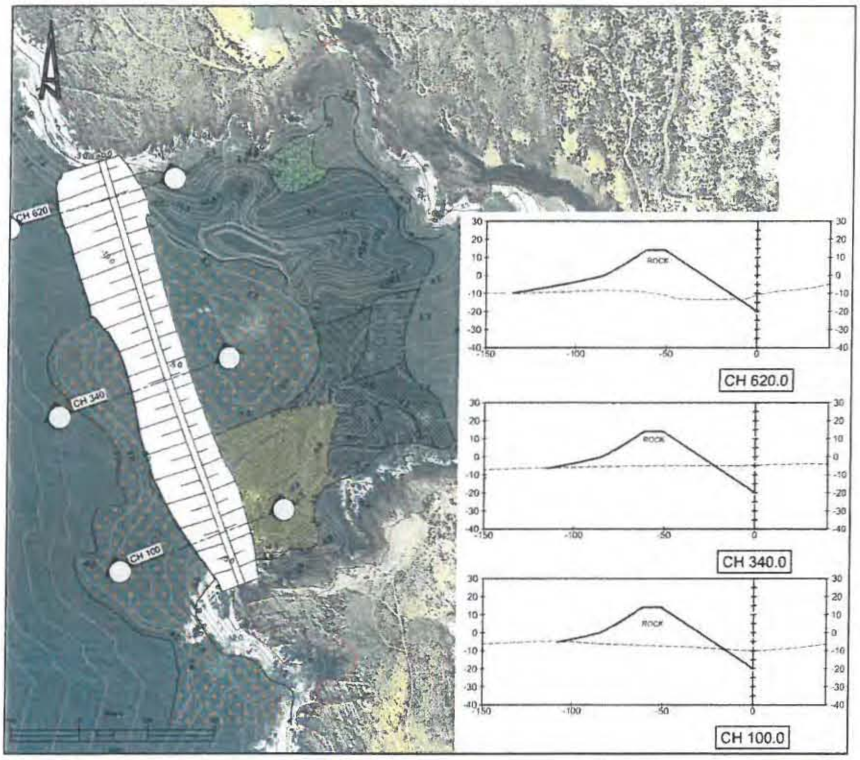

Figure 5.5: Layout and section for dynamically stable berm for closure of Rooiwal Bay

5.3 Reclamation of Bay with Sand

5.3.1 Approach

Possible mining of inland areas at Rooiwal Bay, specifically at the dunefield, will require stripping of large volumes of fine sand overburden material. This material can be used to accrete the beach and reclaim mining area inside the bay. Mining of this accreted area would liberate further material that can be dumped into the sea to gain additional accretion.

5.3.2 Layout

Sand placed in the bay would be re-distributed by currents. Placing sand on the beach would result in the shoreline accreting, with the new shoreline being in equilibrium with wave-driven current (longshore currents). The accreted shoreline would therefore be oriented approximately perpendicular to the oncoming waves. This would hold for accretion resulting from dumping of sand from trucks, or resulting from hydraulic discharge from a dredger.

Three stages of beach accretion were considered – see Figure 5.6 – with the shoreline moved seaward by 150 m for each successive stage.

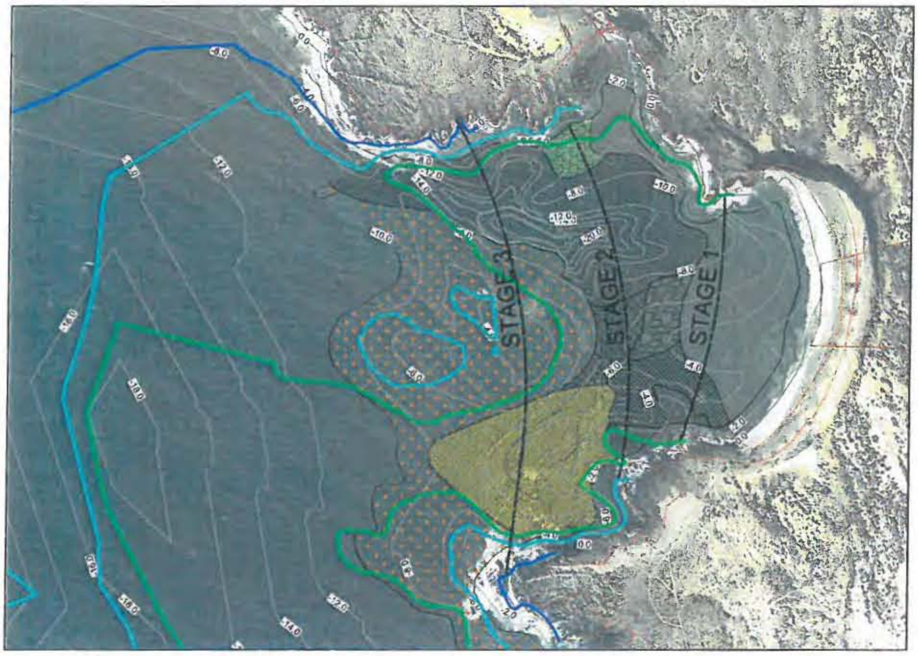

Figure 5.6: Accretion of the shoreline with sand - Stages 1, 2, 3 - and associated deposition of sand on the seabed - green, cyan, and blue lines

5.3.3 Beach Section

The filled beach will assume a dynamically stable cross-sectional profile that is in equilibrium with the wave forces. The shape of the profile will change in response to erosive and accretive wave conditions. The slope of this profile will determine the volume of sand required to fill the bay, and how far the toe of the profile will extend outside the bay. If the toe of the profile extends too far out of the bay, then sand may "leak" from the profile and be transported away from the site, along the coast. This would require that the profile is continually replenished with additional sand in order to maintain the desired "new" shoreline position.

A method that is used to determine the equilibrium profile of a beach is that based on the sand particle size (Dean, 1977). This simplified method provides a first estimate of what the beach profile would look like. The profile calculated using typical fine sand – material from the Site 68/69 beach was used – is shown in Figure 5.7 below.

Figure 5.7: Equilibrium profile for fine sand

5.3.4 Volumes Required

The following volumes of sand fill were calculated, without taking into account any losses:

- Stage 1 – 150m accretion: 1.3 million cubic metres
- Stage 2 – 300m accretion: 2.5 million cubic metres
- Stage 3 – 450m accretion: 5.9 million cubic metres

These volumes are the accretion volumes required. They do not account for re-stripping of material, for example to mine inside the accreted bay during Stage 2 some of the sand placed for Stage 1 would be re-used.

The sand would extend on the seabed seaward of the bay. As the accretion distance increases, progressively more sand would be transported out of the bay and become deposited on the seabed and adjacent coastline. This would also result in the rate of accretion reducing as the accretion distance increases, even though the rate of sand discharge/dumping/placement remains the same.

5.3.5 Rehabilitation

It is likely that a significantly wider, than pre-mining, beach will remain within the bay once mining is completed. However, as the mining would involve excavation of a large pit inland of the accreted beach, breaching of this accreted beach will allow the sea to flood the pit and return the shoreline to a more easterly position. The position of this new post-mining shoreline would depend on the layout of the mining pit. Sand would remain on the deeper areas of the seabed for some time, until wave action has re-worked it towards the shore.

During the accretion process, sand would be dispersed by wave action along the coastline outside the bay. This would increase the amount of sand on what is a predominantly rocky coastline. It would not be feasible to remove this sand.

6 Beach Mining Solutions for Site 68/69

6.1 Proposed Mining Areas

The proposed mining area is located approximately in the centre of the bay, see Figure 2.6. The position of the blocks is indicative at present, as no actual diamond sampling information is available seaward of the HWM. The mining area follows a paleo-channel that enters the bay from the north and north-west.

It is possible that mining may extend up to 300 m seaward of the HWM as consecutive blocks are mines. Typical blocks would have dimensions of 100 m by 100 m.

Geological data and previous mining operations indicate that the diamond bearing gravels are located at approximately -20 m MSL. The gravels are overlaying by some 15 m of clay. The remaining, upper, overburden is sand. The overburden and gravel levels seaward of the HWM are not known. However, they are assumed to be similar. The mining excavation will thus be deep.

6.2 Beach Accretion with Sand

The initial concept for Site 68/69 had been to use sand to accrete additional mining area. However, a previous study (WSP, 2011) indicated that large volumes of sand would be required. Due to high longshore sand transport rates, sand placed for accretion is rapidly transported away. Insufficient sand is available at the site to develop and maintain anything more than limited accretion.

Using sand to accrete the beach for future mining operations is therefore not a viable option at Site 68/69.

6.3 Rock Berms

Rock berms are an alternative means to reclaim mining area into the surfzone and protect the area while stripping and mining occur. Rock material is available near Koingnaas, in stockpiles left over from previous mining operations, although quantities and rock sizes and properties are not known.

6.3.1 Approach

The procedure for constructing a protective rock berm can be described simplistically as follows:

- A rock berm is constructed into the surfzone on both the northern and southern side of the planned mining area. Material is placed by end-tipping from trucks. A dozer and excavator are used to place the material and shape the profile;
- The berms are oriented perpendicular to the oncoming waves and shoreline. In this way only the tip of the advancing berm is exposed to high wave energy;
- The berms extend seawards until the seaward extent of the mining block is reached. At this location, a berm is constructed parallel to the coastline to link the two shore-perpendicular berms;
- The mining block is then enclosed within the berm, with access onto the berm from the shoreline;
- The berm extends landward to above the storm HWM in order to prevent wave runup on the adjacent beaches from flooding the mining area.

The mining would likely progress seaward in stages: the most landward block would be mined first; then the berms would be extended seaward to enclose the next block, which would be mined. This sequence would continue until the most seaward block has been mined. For the purposes of this study, four stages are assumed:

1. Stage 1: Mine to HWM
2. Stage 2: Mine to 100m seaward of HWM
3. Stage 3: Mine to 200m seaward of HWM
4. Stage 4: Mine to 300m seaward of HWM

6.3.2 Berm Design

The plan layouts for Stages 1 to 4 are shown below. Large format drawings of the layouts are provided in Appendix 1. The berm required for Stage 1 would be comparatively small. The toe (most seaward intersection of the berm slope with natural seabed) of the berm would be located at approximately -1 m MSL. No bathymetry information is available in the surfzone, and therefore depths in this area have been estimated. The shallowest depths reached by bathymetry survey undertaken in May were approximately the 5m depth contour. The berms for Stages 2 to 4 would extend into the surfzone and beyond, into progressively deeper water.

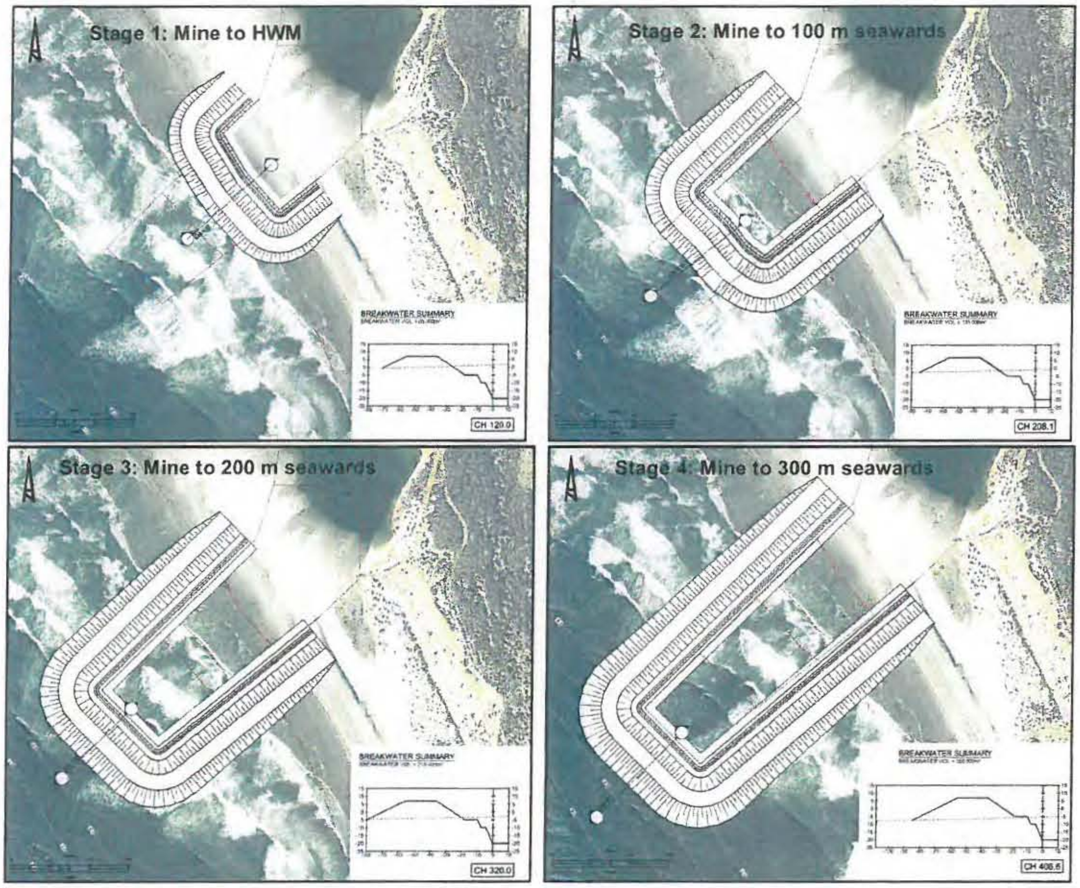

Figure 6.1: Plan view of berm layouts for mining at Site 68/69

Estimated total rock volumes (armour, underlayer, and core material combined) are as follows:

Stage 1: 65 000 m^3

Stage 2: 135 000 m^3 (additional 70 000 m^3 after Stage 1)

Stage 3: 216 000 m^3 (additional 81 000 m^3 after Stage 2)

Stage 4: 356 000 m^3 (additional 149 000 m^3 after Stage 3)

Loss of material would occur from the exposed slopes during construction. This is not included in the above estimates, but would need to be allowed for when determining the volumes required for construction. Additional armour stone sized material would also need to be kept in a stockpile near the site for maintenance of the berms after construction.

The calculated size (mass) of armour stones required are summarised in the table below, based on estimated water depths at the seaward toe of the berms.

Table 6.1: Estimates of required rock size

Berm	Water depth at toe (m to MSL)	Estimated mass of armour stone required (tonnes)
Stage 1 – HWM	-1 (estimated)	0.2 to 1 (median 0.5)
Stage 2 – 100m	-2.5 (estimated)	1 to 3.5 (median 2)
Stage 3 – 200m	-5 (from surveyed depths)	4 to 16 (median 8.5)
Stage 4 – 300m	-8 (from surveyed depths)	5.5 to 21 (median 11)

The sizes are calculated using the method of Van Gent (2004 and 2005) and the Rock Manual (CIRIA, 2007) for conventional statically stable rock berms. The following are assumed in the calculations:

- Design water level is +1.4m MSL
- Slope of seaward face of berms is 1:2 (slightly flatter than repose angle)
- Rock relative density of 2.65 tonnes/m^3.
- Damage to the structure is permissible during mild to severe storms and frequent maintenance would be undertaken.
- The structure would have a comparatively fine core (to reduce seepage).
- The design life of the berm is short – 1 to 2 years. It can therefore be considered as a temporary structure.

Depth limited wave conditions are assumed for Stages 1 to 3. The Stage 4 berm would extend beyond the surfzone and would be subject to unbroken waves. The 1:5 year return period nearshore wave height as determined from the wave refraction study was used for this case.

The berms for Stages 3 and 4 would require very large armour stones that would be difficult to quarry, transport and place. This reduces the feasibility of these structures. A flatter design slope could be considered for Stage 3 in order to allow smaller rock to be used. However, a conventional rock berm solution is not considered feasible for Stage 4.

Possible alternatives for Stage 4 are:

1. Using concrete armour units on the seaward face of the berm;
2. Constructing the berm as a dynamically reshaping profile, as discussed for Rooiwal Bay (Section 5.2.4).

Concrete armour units would be very expensive. It is unlikely that the cost would be justifiable. An experienced marine contractor would be required, as concrete armour slopes require careful construction. Specialised construction plant, such as crane with sufficient capacity to lift and place units, would also be required.

A dynamically reshaping profile would allow smaller rock to be used. However, very high rates of erosion would occur at the corners of the berm as these areas are unstable. It is possible that the rate of erosion will exceed the rate at which material can be placed, with the result that the berm cannot be constructed completely.

7 Generic Berm Designs

A generic design is required for berms or coffer dams that would be used to protect small beach mining operations. These mining operations could be employed at any location within West Coast Resources's mining area. The berms need to resist wave attack and prevent flooding of the mining excavation.

Design of coastal protection measures needs to take into account the particular characteristics of a site. Applying a generic design without adapting it to the site conditions can result in an over-designed structure (high cost), or an under-designed structure that has a probability of failure.

The conceptual designs given in this section are thus described in general terms that would need to be refined for a particular site. The following assumptions are made:

1. The mining (reclamation) areas are small. West Coast Resources have indicated areas up to 200 m by 200 m would be mined at a time.
2. Mining will occur quickly (<6 months duration)
3. Life of protective berms is short (<1 year)
4. Mining excavations are shallow (<5m below NGL)
5. The mining areas are confined to the inter-tidal area and the surfzone, and do not extend to water depths greater than 4m.
6. Materials and equipment are on hand for maintenance of the berms after storms.

The function of the berms is essentially the same as those for Site 68/69 and Rooiwal Bay. A similar design approach was used. Rock would be the only suitable construction material.

7.1 Berm Section

The berms would be located in shallow water (in general terms: within the surfzone), where the greatest wave heights are determined by the local water depth. The method described in the Rock Manual (CIRIA, 2007) was used to determine depth-limited design wave heights for several possible berm locations/water depths.

A wave peak period of 16s was used. The design water level was taken as +1.4m to MSL (Section 2.2).

The slope of the seaward face of the berms was taken as 1:2. This is flatter than the natural angle of repose of rock. However, the stability of rock increases for flatter slopes, allowing smaller rock to be used. The smaller stones are easier to handle and transport. A flatter slope does have the following disadvantages:

- The total volume of rock is greater.
- Greater care needs to be taken to place the rock to the correct slope below water.
- The toe of the structure is located far out from the crest, requiring excavators with very long reach, or a crane. The problem increases as water depths become greater.

7.1.1 Rock Sizes

The berm is designed as a statically stable structure, with an armour layer consisting of two layers of armour stones, placed on an underlayer of smaller stones. The core would consist of finer stones and would be impermeable, or of low permeability, in order to reduce seepage into the mining excavation. The density of rock was taken as 2650 kg/m^3.

The calculated sizes of the armour layer rock are given in **Table 7.1** for structures located at different water depths.

Table 7.1: Calculated armour stone masses

Case	Seabed level at toe (m to MSL)	Design wave height, H_{m0} (m)	Mass of armour stones required (tonnes)
1	-1	1.8	0.4 to 1.7 (median 0.9)
2	-2	2.5	1.1 to 4.5 (median 2.3)
3	-3	3.3	2.7 to 10.3 (median 5.3)
4	-4	3.7	3.8 to 14.6 (median 7.5)

7.1.2 Crest Level

The level of the berm crest must be high enough to limit waves washing over, or splashing over, the berm. Termed overtopping, this could have the following impacts on the mining excavation:

- Flooding, if overtopping is severe, endangering personnel and damaging equipment
- Erosion of the landward (lee) slope of the berm
- Damage or risk to machinery or personnel on the berm crest.

The overtopping rate is affected by several factors: wave and water level conditions, freeboard (defined as the height of the crest above SWL), berm slope angle, roughness of the slope, seabed depths seaward of the berm. Coastal engineering design guidelines (Eurotop, 2007) indicate the following as acceptable limits of overtopping discharge:

1. For personnel on the berm: 1 to 10 litres per second per metre along the berm (l/s/m) for personnel that are trained, well shod and protected, and expecting to get wet;
2. For vehicle on the berm: 10 to 50 l/s/m for vehicles driving at low speed, pulsating flows at flow depths, no falling jets, vehicles not immersed.
3. Damage to defence crest or rear slope: < 50 l/s/m for a well-protected rear slope.

The above discharge numbers exclude spray blown over the berm by onshore winds.

Flooding is likely to be the greatest risk. The overtopping flow rate that is likely to present a flooding risk is not clear, therefore the upper overtopping limit (50 l/s/m) is considered not acceptable. The required berm crest elevations for limits of 1 l/s/m and 10 l/s/m were calculated and are given below.

Table 7.2: Berm crest levels for overtopping rates

Case	Seabed level at toe (m to MSL)	Design wave height, H_{m0} (m)	Crest elevation giving 1 l/s/m overtopping (m to MSL)	Crest elevation giving 10 l/s/m overtopping (m to MSL)
1	-1	1.8	4.6	3.6
2	-2	2.5	6.1	4.7
3	-3	3.3	7.9	6.1
4	-4	3.7	8.9	6.8

The crest width would be designed for operational factors, primarily to allow two-way traffic for trucks and dozers. A width of 10m is assumed here.

7.2 Berm Layout

No specific site has been defined that would require a berm as outlined above. A possible layout is indicated in the figure below for a situation where the mining site is located in a bay, with rocky headlands on either side (a large format drawing of the berm is also given in Appendix 1). The berm could be constructed outward from the headland, before turning parallel to the coast. This connects to the shore with a shore-perpendicular berm.

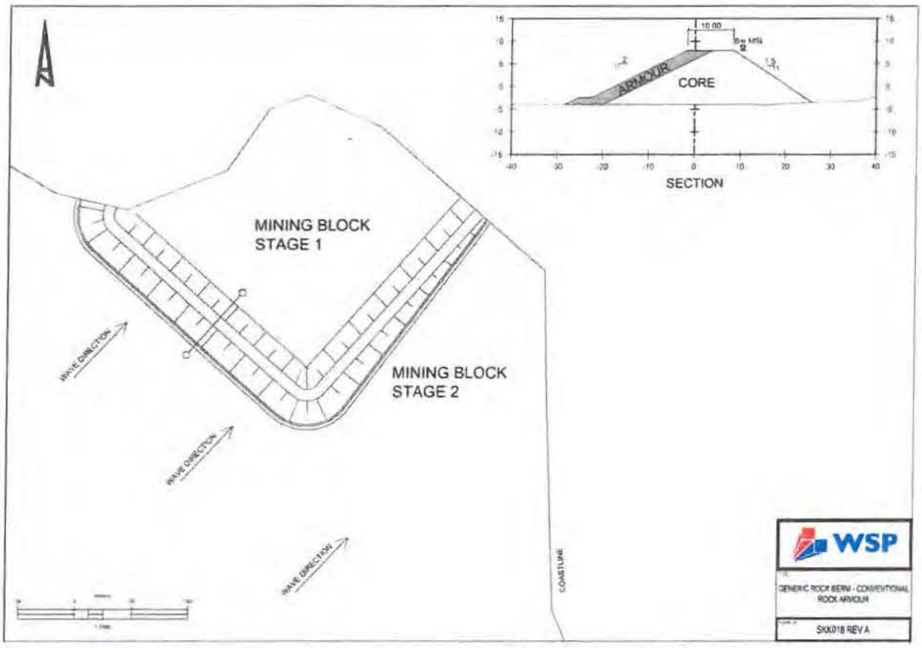

Figure 7.1: Layout of generic rock berm with conventional statically stable rock armour slope

The cross-section design, and thus required rock volumes, would be a function of the water depth at the specific site.

The mining could occur in stages, with additional berms constructed or extended as additional mining area is reclaimed. It may be possible to re-use berm material by stripping it from the areas where mining has been completed and using it to construct new berms. However, removing large armour stones from below the water line can be difficult and risky for machinery and personnel.

7.3 Alternative Berm Section

The statically stable rock berms outlined in the previous section require large armour stones for berms located in water deeper than -3m MSL, e.g. median mass of 5.3 to 7.5 tonnes for water depths of 3 m to 4 m. Such heavy stones may be difficult to produce in sufficient quantities, as well as requiring large and specialised machinery to load, transport, and place. This reduces the feasibility of such a berm.

An alternative would be to construct a dynamically reshaping berm, similar to what was described for Rooiwal Bay. This concept is illustrated in Figure 7.2 (a large format drawing of the berm is also given in Appendix 1).

The reshaping berm would be applied in a shore parallel direction, such that it is approximately at right angles to the wave direction. The berm constructed perpendicular to the shore – the groyne – would need to be a conventional statically stable armour rock structure in order to avoid loss of material from oblique wave attack.

The seaward end of this groyne, the head, would require large armour stones such as calculated in Table 7.1 for the appropriate water depth. The size of armour stone along the trunk of the groyne could be reduced as the water depths decrease towards the shore.

The dynamic reshaping berm needs to be sufficiently wide (thick), particularly as it will take several storm events in order for a stable shape to be attained. After initial construction, where material would be end-tipped at the angle of repose, each subsequent high wave and water-level event will shape the profile. During this time it may be necessary to place additional material in order to maintain a safe crest width and elevation. For a berm located in water depths of 3m to 4m to MSL, the crest elevation of between 11m and 12m would be required.

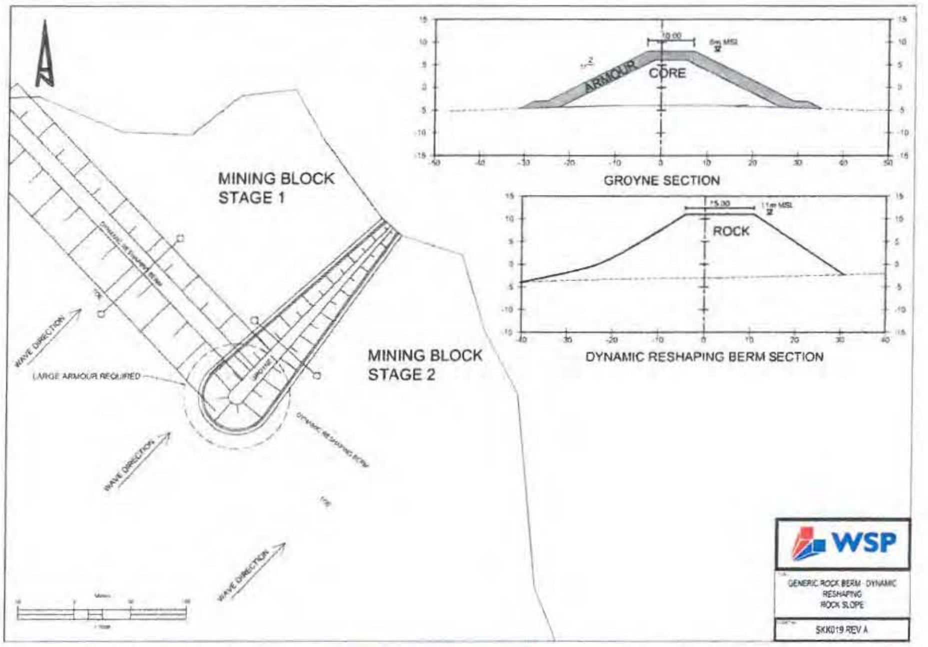

Figure 7.2: Layout of alternative berm - dynamic reshaping rock slope with shore perpendicular groyne

7.4 Rehabilitation

Berm material can be re-used from areas where mining has been completed. However, it is unlikely that material located in deeper water can be retrieved economically and this material would remain on the seabed. Wave action would rapidly move and sort the remaining material, particularly the smaller core fraction.

The design life of the berms is relatively short, and thus over a period of a few years they would be eroded by large storms. This process would be speeded up by removing the larger armour stones and lowering the crest level.

8 Conclusions and Recommendations

8.1 Rooiwal Bay

The northern channel in Rooiwal Bay is partly sheltered from severe wave energy. It will be feasible to construct a conventional, statically stable rock berm across the mouth of this channel. However, the mining area gained will be comparatively small.

Three coastal protection options have been considered that would allow conventional open-cast mining of the greater part of the bay:

1. Closing off the entire bay with a conventional, statically stable, rock berm breakwater. Engineering design calculations indicate that this option would only be feasible if concrete armour units are used. These are likely to be too costly. This option is therefore not recommended.
2. Closing off the entire bay with a dynamically stable rock berm. Construction is likely to prove difficult due to high loss rates of rock off the partly completed berm. This option is considered costly but technically feasible.
3. Reclamation of the bay area with sand. This option is considered feasible. It is likely that costs will be lower than for the other options. However, this option is dependent on the mining of the inland deposits for a source of reclamation material. It is recommended that this option is considered further.

A combination of measures could be considered for mining of the greater bay area. For example, a rock sill (submerged mound of rock), or a small berm, could be placed in order to partially fill the deep channel in the north of the bay. This would restrict the movement of sand out of the bay. The feasibility of this option should be investigated further.

8.2 Site 68/69

Four stages of mining were considered, with rock berms used to enclose mining areas that progressively extend further seawards. The feasibility of Stages 3 and 4, where mining would extend 200 m and 300 m seaward of the HWM respectively, is considered questionable. This is as a result of the deep water that would be encountered at the seaward toe of the berms as they extend into and beyond the surfzone: -5m and -8m to MSL for Stages 3 and 4 respectively.

It is considered possible to design and construct an appropriate rock berm to allow mining up to Stage 3. To mine up to Stage 4 will require considerable investment in construction of an appropriate coastal protection structure that would likely require the use of concrete armour units or a dynamically stable rock berm. Construction to both Stage 3 and 4 would require a contractor with marine experience, as well as the required construction plant.

8.3 Generic Berms

Design of generic berms has been investigated. Similarly to Site 68/69, the design is a function of the prevailing water depth, amongst other factors. This determines the size of the rock armour required, as well as the crest elevation needed to limit overtopping.

Conventional statically stable rock berms are recommended for water depths shallower than 3 m to MSL. For deeper water, dynamically reshaping berms, where smaller stone can be used, are recommended. These should be oriented at right angles to the oncoming waves, if possible, in order to limit erosion of material from the berm.

8.4 General

The protective berms discussed for Site 68/69 could also be applied at Somnaasbaai, as the site visit and bathymetric survey have indicated conditions there to be similar to those at Site 68/69.

Detailed design should be undertaken of the preferred coastal protection option for both Rooiwal Bay and Site 68/69. This detailed design should examine the potential for scour and correct design of the berm toe, amongst other factors.

It would be prudent to carefully consider the marine conditions at any site where a "generic" rock berm is to be used for coastal protection. Sites may seem similar under certain wave or tide conditions, but factors such as offshore reefs or differences in seabed slope may result in pronounced differences during storm conditions. Applying a generic design to different sites is thus inherently risky.

Physical modelling of rock berms, particularly those requiring concrete armour units, is recommended.

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8.1.1. Addendum 1: Rock volumes for generic type coastal protection berms

Coastal Protection Options for Beach Mining in the Koingnaas Area

WSP Report No. 19943/11.2/001 Rev01

Addendum 1: Indicative Rock Volumes for Generic Type Coastal Protection Berms. Addendum to Section 7: Generic Berm Designs

Date: 31 August 2015

1. Statically stable rock berm

The layout of the berm is shown in Figure 1 below for a theoretical mining site. Indicative volumes are given in Table 1 for an assumed mining area of approximately 200 m by 200 m and seabed depths up to -2.5 m MSL at the location of the shore-parallel berm. The grey highlighted part in Figure 1 is that required to protect the Stage 1 Mining Block. The lighter shaded berm is the additional shore-parallel berm required for protection of the Stage 2 Mining Block.

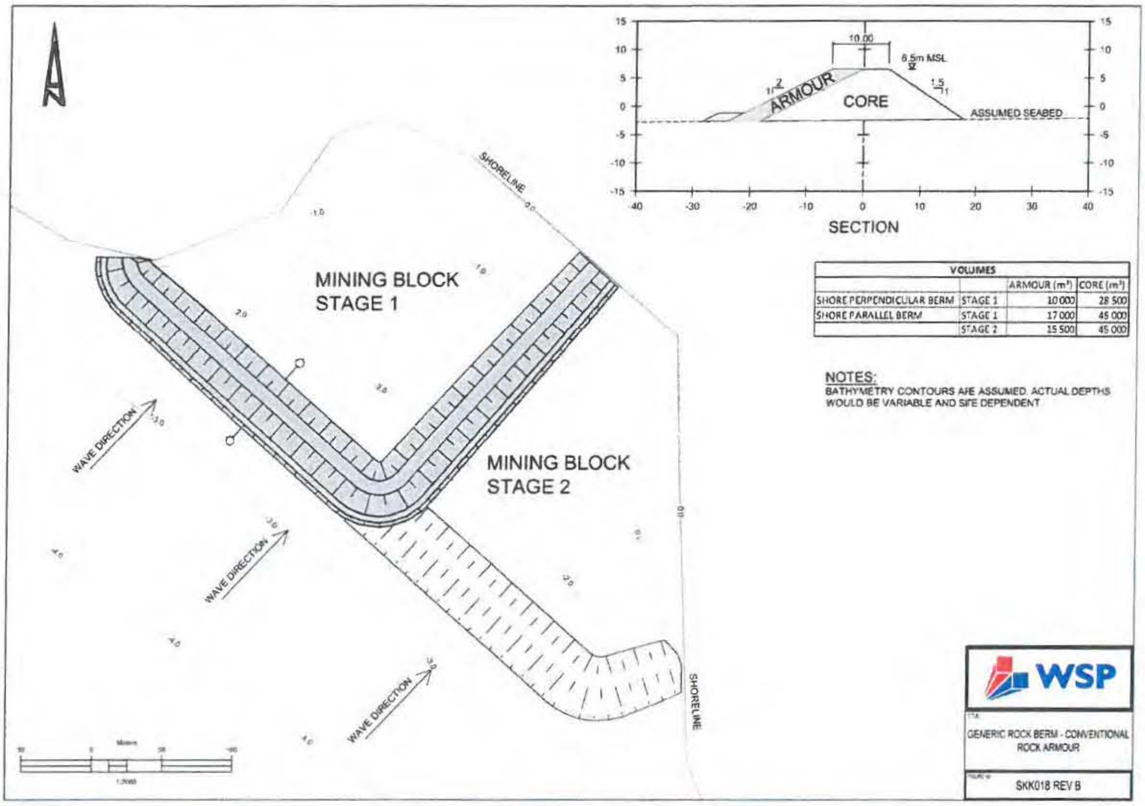

Figure 1: Layout of generic rock berm with conventional statically stable rock armour slope

Table 1: Indicative volumes for conventional berm

		Volume (m^3)	
		Armour	**Core**
Stage 1 Mining Block	Shore-perpendicular berm	10 000	28 500
	Shore-parallel berm	17 000	45 000
Stage 2 Mining Block	Extension of shore-parallel berm	15 500	45 000

2. Alternative Berm

The layout for an alternative, dynamic reshaping type berm is shown in Figure 2. Indicative volumes are given in Table 2 for an assumed mining area of approximately 200 m by 200 m and seabed depths up to -4.0 m MSL at the location of the shore-parallel berm. The protection works consist of the shore-perpendicular groyne, constructed as a conventional rock structure, and a dynamic reshaping shore-parallel berm. The berms required for Stage 1 Mining Block are highlighted in grey. The additional shore-parallel dynamic reshaping berm required for protection of the Stage 2 Mining Block is shown in lighter shading.

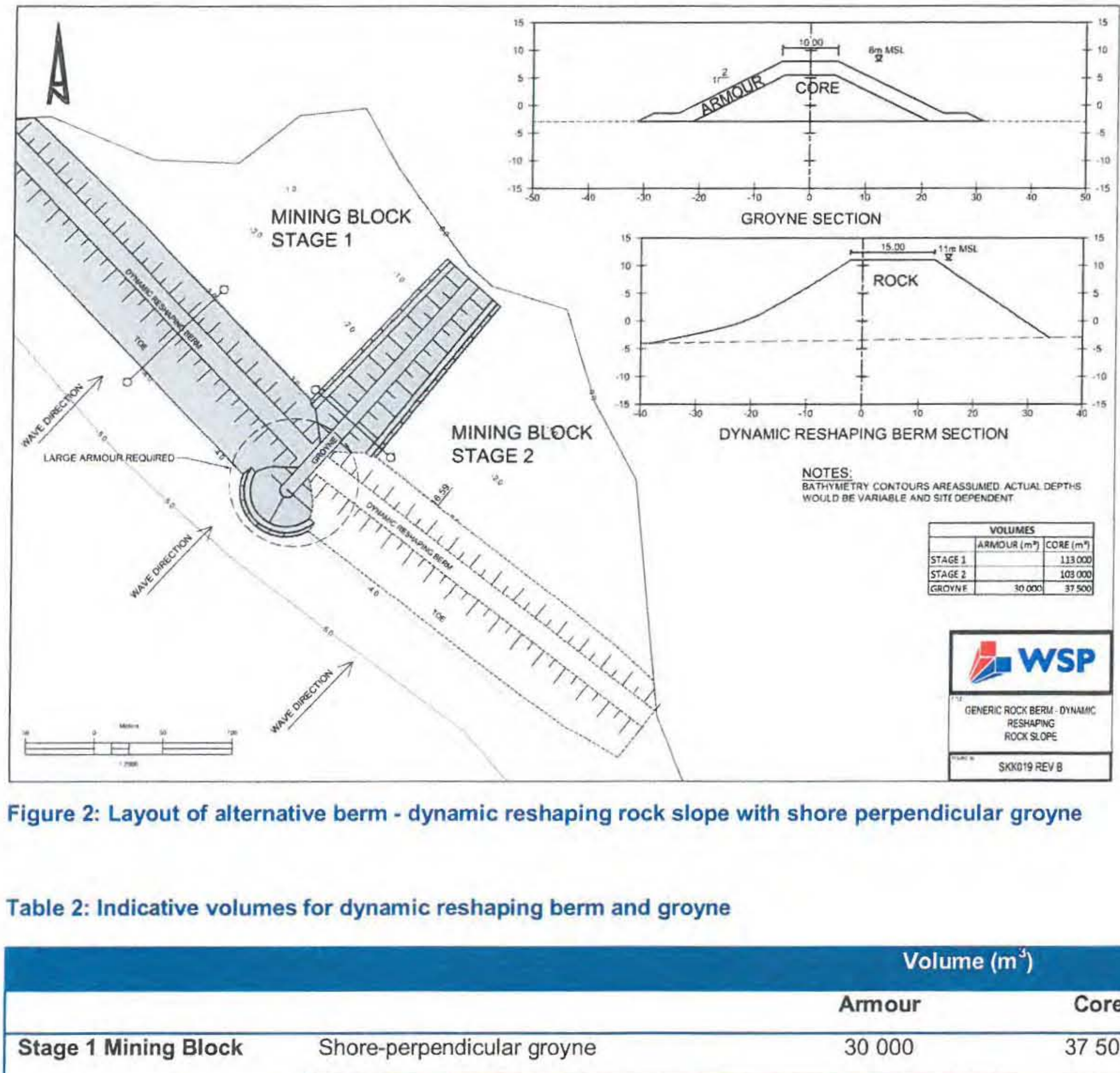

Figure 2: Layout of alternative berm - dynamic reshaping rock slope with shore perpendicular groyne

Table 2: Indicative volumes for dynamic reshaping berm and groyne

		Volume (m^3)	
		Armour	**Core**
Stage 1 Mining Block	Shore-perpendicular groyne	30 000	37 500
	Shore-parallel dynamic berm	113 000	
Stage 2 Mining Block	Additional shore-parallel dynamic berm	103 000	

8.1.2. Addendum 2: Development of additional beach mining options