Port Spencer Public Environmental Report
Sediment Transport Modelling

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Report prepared for Golder Associates (Adelaide)

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Cover Picture: View of the shoreline near the vicinity of the Project.

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EXECUTIVE SUMMARY

This report reviews the oceanographic environmental conditions at the proposed location of Port Spencer located in the Spencer Gulf, South Australia and relates them to potential disruptions in sediment transport as a result of construction of a jetty for the loading of ore and grain transport vessels.

Due to the level of development involved in this project, regulations require that the following specific topics be addressed:

- extreme wind, wave and tidally driven currents at the Project
- quantification of existing flow regime
- existing sediment transport regime at the Project
- scour effects due to jetty and associated piling
- changes in flows due to the proposed development
- how the Project may alter sediment transport
- potential changes to beach profiles.

The analysis for these criteria began with a review of the data collected and analysed in previous studies. Starting with the water level, it was shown that the majority of the water level fluctuations experienced at the Project resulted from the tide, with smaller water level changes associated with wind and wave induced set up, barometric pressure effects and coastal trapped waves. The overall tidal range is on the order of 2.1 m while the other factors combined are on the order of 0.4 m.

Currents at the Project were measured using an Acoustic Doppler Current Profiler (ADCP). Overall, the maximum current speeds were between 0.34 and 0.69 ms^{-1} with larger current speeds observed at the top of the water column. Mean current speeds were 0.14 ms^{-1} at the top of the water column and 0.10 and 0.09 ms^{-1} for the middle and bottom of the water column, respectively.

Waves affecting the Project site are either generated locally by winds which create short period, steep waves or propagate from the Southern Ocean in to Spencer Gulf and reach the Project site as long period swells. During the summer months the predominant onshore south-easterly winds result in a more dominant short period wave climate. Whilst during winter, the winds are predominantly offshore and a more active open-ocean wave climate increases the occurrence of long period swell penetration in the Spencer Gulf. Numerical models based on the observed and recorded environmental conditions were able to reproduce the measured wave heights.

Extreme wave heights were assessed through an analysis of long term wind records for locally generated wind waves and from offshore hind cast wave data for the penetration of open ocean swell waves. Extreme significant wave heights at Port Spencer are estimated at 3.7 to 4.7 m for return periods of 1 to 100 years respectively.

Both summer and winter wave climates were modelled to quantify the potential change in wave induced currents and beach response due to the presence of a vessel at the jetty. Wave heights directly in the lee of a vessel are reduced by an average of 0.4 to 0.7 m. Wave heights directly inshore of a vessel would be reduced by around 0.5 m and this would be offset by a slight increase in wave height to the north and south of the jetty.

The sediment transport regime was assessed by combining the hydrodynamic effects of tidal and wave generated currents. Tidal currents alone are not strong enough to suspend sediment, however wave breaking and wave orbital currents at the seabed can entrain sediment that is then moved by the tidal current. Rather than explicitly modelling the bed level changes over the site we used an approach that yields the volumetric transport rate which is the volume (m^3) of sand moving per metre per unit time (s). This allows an investigation into the degree to which sediment will be mobilised over the model domain without examining pathways of sediment movement. Simulating the same period both with and without a vessel present in the model shows how bed erodibility will change in the presence of a vessel. Results were taken from a three week model run but were scaled to give annual rates of transport.

For both net and gross transport rates the values are consistent outside the breaker zone in the absence of a vessel, with a gross rate of approximately 350 m^3/annum/m and net transport of approximately 50 m^3/annum/m. However in the presence of a vessel, both gross and net transport rates drop to nearly zero in its lee, where very little wave energy penetrates. This is offset by an increase in net transport rates to the south of the jetty structure of around 70 m^3/annum/m. With an occupancy rate of 20%, the change in net transport rates would be of the order of 10-15 m^3/annum/m. Assuming this decrease in transport rate leads to deposition along the length of the berthing jetty it could be expected that the bed level change in the lee of the vessel could range from 0.03 to 0.05 m/annum.

The presence of a vessel at the jetty will result in changes to wave heights directly inshore of it (offset by a slight increase in wave height to the north and south of the Project). An increase in sediment is predicted to build up immediately inshore of the jetty. North of the jetty accretion may be slightly reduced and the area where erosion is currently predicted to be occurring may decrease.

Numerical modelling also showed that the predicted formation of scour holes adjacent to the jetty piles would have a minimal effect on the wave conditions at the Project.

The environmental effects modelled are shown to be relatively localised in relation to the Project and do not extend significant distances up and down coast. For example Lipson Island is located approximately 1.5 km south of the Project, and the modelling undertaken indicated no significant changes in the environmental regime at this location.

To provide more confidence in our assessment, a programme of monitoring should be put in place. It is recommended that bathymetric surveys are established at the Project annually to monitor bed changes in the lee of the jetty. A regular programme of beach monitoring should also be put in place prior to any site construction. It is recommended that regularly spaced beach profiles are established along the length of the adjacent beaches, with surveys to low water carried out at monthly intervals. Data collected over the next twelve months would give a good indication of the existing beach dynamics. In order to monitor changes post construction, quarterly beach profile surveys should be undertaken for at least 3 years. This data could be put in context of longer time frames by an analysis of any historical aerial photographs of the area. If mitigation is required sediment by-passing or renourishment options could then be investigated.

1 BACKGROUND

This report gives details of the modelling and assessment of environmental conditions carried out for the Port Spencer development (the Project) to be developed by Centrex Metals Limited. The Project is located on the Eyre Peninsula, South Australia, approximately 21 km north-east of Tumby Bay (Figure 1.1). Note that the Project was previously referred to as Sheep Hill Port, as can be seen in references to earlier reports.

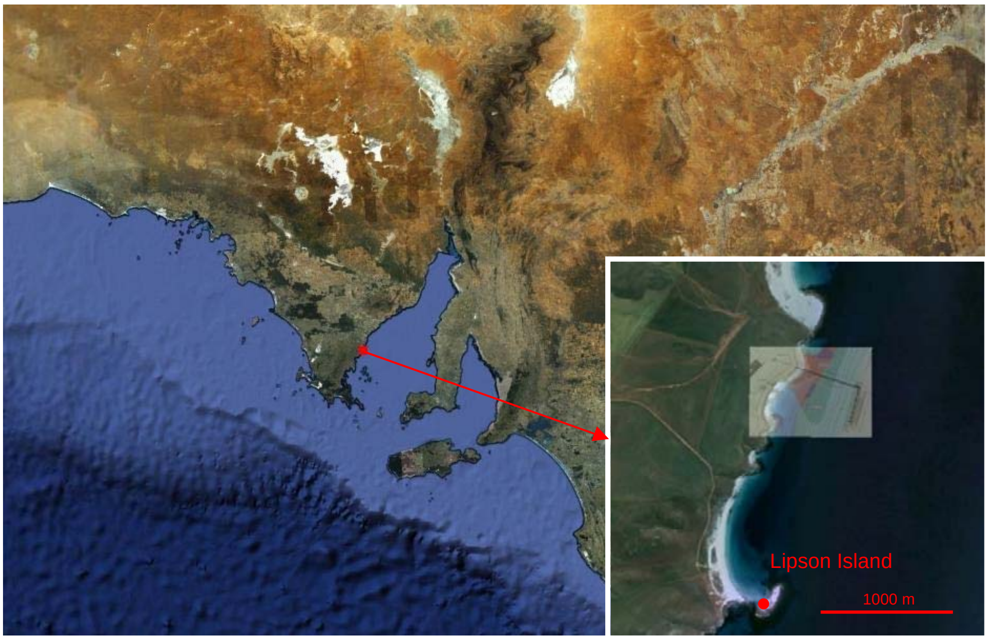

Figure 1.1. The Project location, Eyre Peninsula, South Australia. Inset shows the proposed Port Spencer jetty.

As the Project is considered a Major Development, the Development Assessment Commission (DAC) has prepared a document outlining guidelines for the preparation of a Public Environment Report (PER) (DAC, 2011). The report has been prepared in support of the PER for the Project and addresses environmental issues relating to the following, which relate to Sections 5.3.2, 5.3.3, 5.3.14 and 5.6.7 of the Guidelines:

- extreme wind, wave and tidally driven currents at the Project
- quantification of existing flow regime
- existing sediment transport regime at the Project
- scour effects due to jetty and associated piling
- changes in flows due to the proposed development
- how the Project may alter sediment transport
- potential changes to beach profiles.

This report follows on from earlier reports outlining the field work and modelling carried out in 2009 and 2010 (Grant et al. 2010a, Grant and Moores 2010 and Moores et al. 2010).

A number of instruments were deployed at the Project between 14 October 2009 and 16 March 2010. In addition an additional deployment was carried out between 3 August 2010 and 17 September 2010 to quantify winter conditions at the Project. The deployment site (34.24991 S, 136.27197 E - Figure 1.2) is in approximately 20 m depth of water (Lowest Astronomical Tide). This location was chosen as it is where the jetty will be constructed.

Figure 1.2. Instrument deployment location, Port Spencer.

2 SUMMARY OF RELEVANT DATA

In this section of the report a summary of the instrument data, sediment mapping and Project information relevant to the modelling is presented.

2.1 Water Level Variations

Sea level variations around the site are due primarily to tidal forcing. Smaller fluctuations occur due to winds, changes in barometric pressure and coastal trapped waves. Ansell et al (1997) describe tides in the Spencer Gulf as generally semi-diurnal, with non-uniform phase, and amplitude increasing to the upper estuary with a 6 h phase lag. Loss of semi-diurnal tidal constituents occurs on a regular 14 day cycle, and if the period of low tidal flow coincides with a period of weak winds can result in virtual slack water for 2-3 days (dodge tides).

Table 2.1 gives the amplitude of the eight largest tidal constituents at the Project as derived from water level records recorded between October 2009 and March 2010. This data leads to the estimated tide levels shown in Table 2.2.

Table 2.1. The eight largest tidal constituents from the tidal analysis of the water level record at Port Spencer (Oct 2009- March 2010).

	Symbol	Period (Solar Hrs)	Amplitude	Description
Semidiurnal tides (approximately two tides per day)	M_2	12.42	0.2442	Main lunar semidiurnal constituent
	S_2	12.00	0.3049	Main solar semidiurnal constituent
	N_2	12.66	0.0270	Lunar constituent due to monthly variation in the Moon's distance
	L_2	12.19	0.0322	Smaller lunar elliptic semidiurnal
	K_2	11.97	0.0885	Lunisolar semi diurnal
Diurnal tides (approximately one tide per day)	K_1	23.93	0.2640	Solar-lunar constituent
	O_1	25.82	0.1903	Main lunar diurnal constituent
	Q_1	26.87	0.0522	Larger lunar elliptic diurnal constituent
	P_1	24.07	0.0700	Solar diurnal

Table 2.2. Estimated tidal planes* based on constituent analysis of water level records at Port Spencer (Oct 2009 - Mar 2010).

Tidal Levels	Derivation	Level (m CD)
Highest Astronomical Tide	Sum of constituents	2.52
Mean High High Water	$Z_0+(M_2+K_1+O_1)$	1.81
Mean High Water Spring	$Z_0+(M_2+S_2)$	1.66
Mean Low High Water	$Z_0+abs(M_2-(K_1+O_1))$	1.32
Mean High Water Neap	$Z_0+abs(M_2-S_2)$	1.17
Mean Sea Level	$Z_0^{\#}$ Assumed mean sea level relative to chart datum (CD)	1.11
Mean Low Water Neap	$Z_0-abs(M_2-S_2)$	1.05
Mean High Low Water	$Z_0-abs(M_2-(K_1+O_1))$	0.90
Mean Low Water Spring	$Z_0-(M_2+S_2)$	0.56
Mean Low Low Water	$Z_0-(M_2+K_1+O_1)$	0.4115

* Australian Tides Manual - Intergovernmental Committee On Surveying & Mapping.

[#] Z_0 (mean sea level offset from Chart Datum - CD) assumed to be 1.11 m CD.

There were non-tidal sea level variations observed throughout the record with deviations of up to 0.4 m about mean sea level (MSL) corresponding to a combination of atmospheric pressure changes (0.01 m per mbar deviation from standard atmospheric pressure), wind setup or set down (+/- 0.2 m depending on wind strength and direction) and low frequency sea level variation due to coastal trapped waves and storm surge (+/ 0.3 m).

2.2 Currents

A summary of current velocities recorded by the Acoustic Doppler Current Profiler (ADCP) at the Project between October 2009 and March 2010 is presented in Table 2.3. Overall, the maximum current speeds were between 0.34 and 0.69 ms^{-1} with larger current speeds observed at the top of the water column. Mean current speeds were 0.14 ms^{-1} at the top of the water column and 0.10 and 0.09 ms^{-1} for the middle and bottom of the water column, respectively. Table 2.4 shows the non-tidal and tidal components of the observed currents. Mean currents are less than 0.1 ms^{-1}, maximum tidal currents are less than 0.3 ms^{-1} and maximum non-tidal residuals (0.44 ms^{-1}) occur near the surface.

Table 2.3. Statistics pertaining to the currents* recorded by the ADCP during the field programme at Port Spencer.

	16.33 m from seabed	8.33 m from seabed	4.33 m from seabed
Maximum Easterly current (ms^{-1})	0.51	0.16	0.15
Maximum Northerly current (ms^{-1})	0.53	0.37	0.34
Maximum Speed current (ms^{-1})	0.69	0.37	0.34
Maximum Westerly current (ms^{-1})	0.40	0.15	0.13
Maximum Southerly current (ms^{-1})	0.61	0.28	0.25
Mean Speed (ms^{-1})	0.14	0.10	0.09

* All velocities are presented in ms^{-1} and are in terms of direction current is flowing towards.

Table 2.4. Statistics pertaining to the tidal and residual currents from the analysis of currents recorded by the ADCP during the field programme at Port Spencer.

	16.33 m from seabed	8.33 m from seabed	4.33 m from seabed
Residual Maximum Speed (ms^{-1})	0.44	0.17	0.15
Residual Mean Speed (ms^{-1})	0.08	0.04	0.04
Tidal Maximum Speed (ms^{-1})	0.28	0.26	0.23
Tidal Mean Speed (ms^{-1})	0.09	0.08	0.07

2.3 Waves

Wave heights recorded over the winter deployment were typically less than 1.0 m, but events up to 1.8 m were recorded. Peak wave direction was predominantly from the southeast around 140-150°, with a small component from the northeast at around 70°. In contrast to the summer deployments, winter peak wave periods were typically higher with a mean of 9 s compared to 4 s. During the summer months the predominant onshore south-easterly winds result in a more dominant short period wave climate. Whilst during winter, the winds are predominantly offshore and a more active open-ocean wave climate increases the occurrence of long period swell penetration in the Spencer Gulf.

Two approaches were used in the previous reporting (Grant et al. 2010a) to quantify the wave climate at the Project. The first approach used local winds to predict the shorter period waves which dominate the summer wave climate. A comparison of measured and modelled results is shown in Figure 2.1 which indicates that the observed variations in significant wave heights and peak periods are reasonably well modelled. Excluding periods of offshore winds and larger waves (driven by ocean swell penetration), a linear regression of measured and modelled results shows that 60% of the observed variance in wave height can be accounted for by the predicted wind generated waves and that the model consistently under predicts significant wave height by around 0.1 m. Thus for smaller waves the model becomes less accurate but for larger wind driven waves the model can reproduce the waves to within +/- 5%.

The second approach compared the high period waves measured at the Project with modelled oceanic swells outside the gulf entrance. The modelled wave data is taken from an archive of WW3 model output. WW3 is a third generation wave model developed by the US National Oceanic and Atmospheric Administration / National Centers for Environmental Predictions. For both the summer and winter wave records, a reduced amount of energy enters the gulf in the form of long period swell. On average the significant wave height (Hs) of oceanic swell arriving at the Project is reduced by 80%.

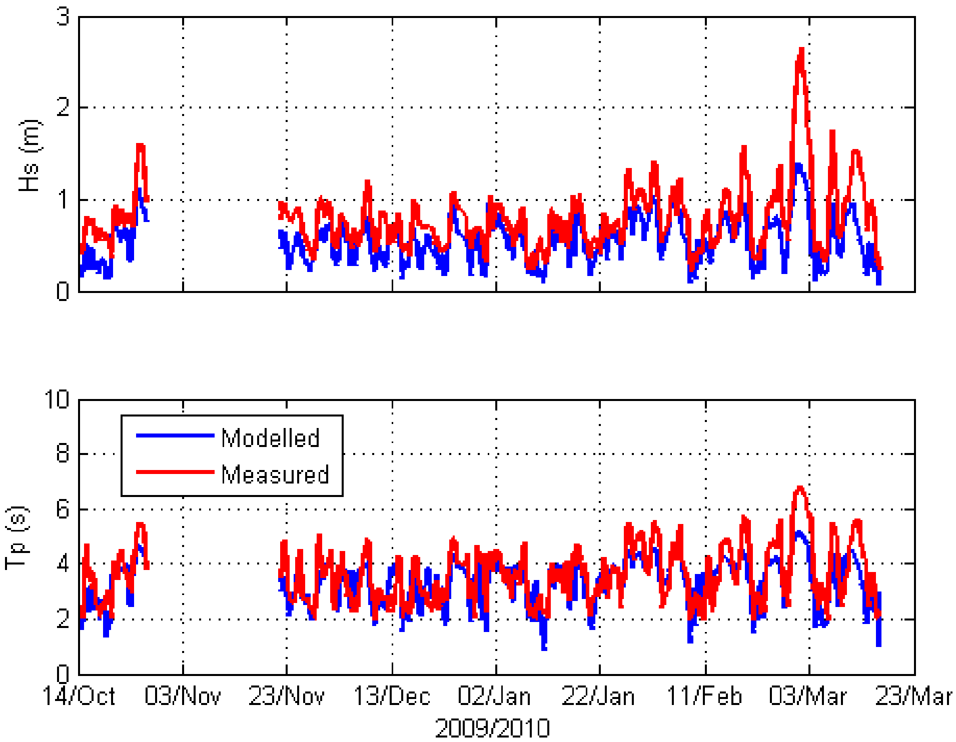

Figure 2.1. Measured versus modelled data comparing significant wave height (Hs) (upper panel) and peak wave period (Tp) (lower panel). The model only considered waves generated by winds between 225° and 45° True North, as waves generated by winds outside of this range are negligible since they are generated by offshore directed winds.

2.4 Temperature and Salinity

Salinity measurements at the surface were typically 32 to 34 practical salinity units (psu), whilst at depth salinity varied between 34 and 38 psu. Other significant variations in salinity and stratification within the water column were not evident during the field programme.

Water temperatures showed strong diurnal variation due to the influence of solar heating and night time cooling. This effect is more apparent at the surface than at depth and gives rise to a diurnal thermal stratification. Stratification in the water column was most apparent during prolonged periods of high air temperature, relatively light winds and neap tides (i.e. periods when vertical mixing would be minimal and stratification of the water could be expected).

2.5 Sediment Mapping

A geophysical survey was carried out on 19 and 20 September 2011 and provides maps of the distribution of unconsolidated sediments in the vicinity of the proposed jetty (Golder Associates, 2010). Along the length of the berthing wharf a layer of medium to fine grained sediments occur. Bedrock occurs at approximately 1 m below the seabed in the area of the proposed jetty. Along the length of the approach jetty a maximum sediment thickness of around 5 m occurs approximately 200 m from shore. Either side of this maximum the sediment thickness tapers off to approximately 1 m. Mean sediment grain size for the area around the berthing wharf is 0.13 mm. Mid-way along the approach wharf mean sediment grain size increases to 0.30 mm, suggesting sorting of sediments due to wave mechanism as observed in open coast situations (e.g. Black and Oldman, 1999).

A rocky reef extends 50 to 70 m offshore either side of the headland inshore of the jetty structure (Figure 2.2). To the south and north of the Project a number of rocky headlands exist along with pocket beaches ranging in width from 20 m through to 180 m wide.

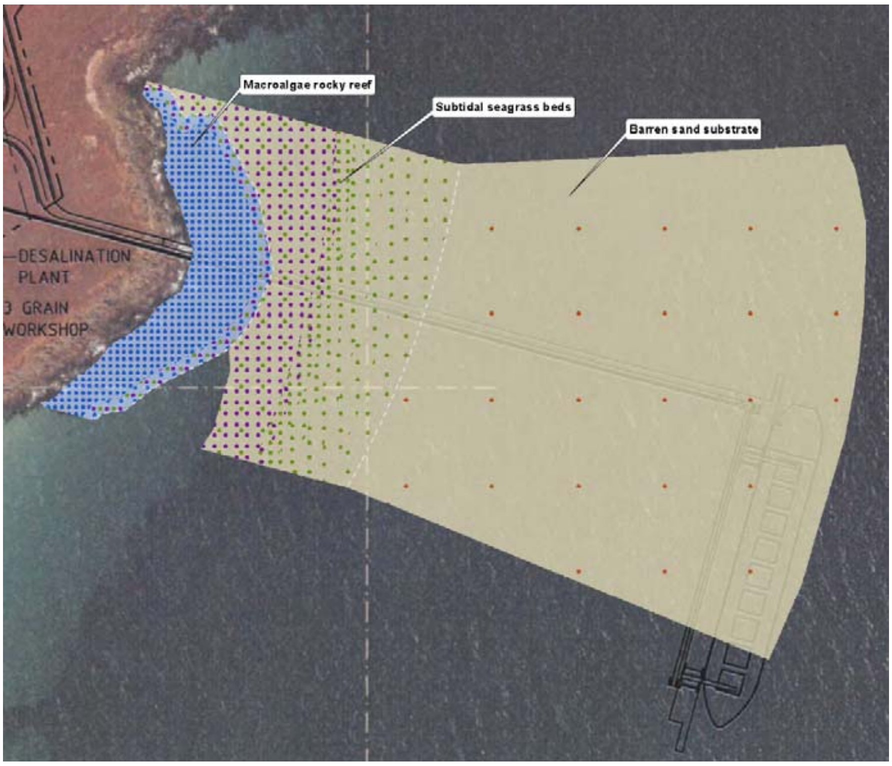

Figure 2.2. Subtidal habitat map inshore of the proposed jetty (Source: Figure 2, Golder Associates, 2011).

2.6 Percentage Time Vessels Will Be On Site

To determine the effects of the Project, an estimate of the length of time vessels may be moored at the site has been used. This has been based on information provided by Golder Associates as described below along with estimated export volumes from various Project development stages (Table 2.5).

Either a Cape class vessel (for ore) or Panamax vessel (for grain) would be moored at the jetty at any one time. The loading rates are 5,000 tonnes per hour for ore and 2,500 tonnes per hour for grain. The iron ore storage facilities for the Project will be 165,000 tonnes. At the specified loading rate, a Cape class vessel would be loaded in three days and there will be 12 ore vessels visiting per year to carry the proposed export of 2,000,000 tonnes per year. The grain storage facilities for the Project will be 60,000 tonnes, so at the specified loading rate, a Panamax vessel would be loaded in one day and there will be eight vessels per year to carry the proposed export of 500,000 tonnes per year. Additionally it has been assumed at least one day additional to the loading time would be needed to allow for customs and quarantine inspections.

Thus moored days would be a total of 48 for Cape Class vessels (i.e., three days loading plus one day customs/quarantine, twelve times per year) and 16 days for Panamax vessels (one day loading plus one day customs/quarantine, eight times per year). This equates to approximately 20% of the year.

Table 2.5. Estimated export volumes at various Project stages (Source: Golder Associates).

Stage	Timing	Grain (road / possible future rail)	Hematite (road)	Magnetite (pipeline)
1	Late 2013	500,000 tpa	Up to 2,000,000 tpa	0
2	Late 2015	Up to 1,000,000 tpa	2,000,000 tpa	5,000,000 tpa
3	tbc	1,000,000 tpa	2,000,000 tpa	10,000,000 tpa
4	tbc	1,000,000 tpa	2,000,000 tpa	Up to 20,000,000 tpa

3 EXTREME WAVE ANALYSIS

In this section an estimate of the extreme waves that are likely at the site due to both ocean swell and locally wind driven waves is presented.

3.1 Open-Ocean Swell Penetration

A SWAN model (Cycle III version 40.51) was developed in order to simulate open-ocean wave penetration into the Project and to validate the 80% reduction in offshore to inshore wave heights derived in earlier reporting (Grant et al., 2010a). The model domain covers the area between latitudes 32° and 36° South, and longitude 135° and 139° East, with a grid spacing of 0.02°. The model is driven using the ASR MDI database, an archive of WW3 model hindcast output dating back to 1997. WW3 is a third generation wave model developed by the US National Oceanic and Atmospheric Administration / National Centers for Environmental Predictions. WW3 provides wind and wave model boundaries at 0.5° resolution (81 boundary nodes) for the inclusive 6 year period February 2005 to July 2011.

The ratio of predicted wave heights at the Project compared to the open-ocean significant wave heights (36° S 135° E) are shown in Figure 3.1. For the larger waves at the site a ratio of 0.22 occurs which is in good agreement with the one-fifth value derived earlier - where offshore wave heights from the WW3 model were compared to the observed wave heights from the deployment programme.

An extreme wave height analysis was undertaken for the full 14 year ASR MDI record (1997-2011) for the open-ocean node located 36°S 135°E (Table 3.1). Non directional wave heights for return periods from 1 to 100 years range from 9.3 m to 12.3 m, respectively. Assuming that open-ocean wave heights are reduced to 22% of these values, then significant wave heights from 2.05 m to 2.71 m could be expected at the site for return periods from 1 to 100 years. This does not take into account locally generated waves within the gulf that are addressed separately in the following section.

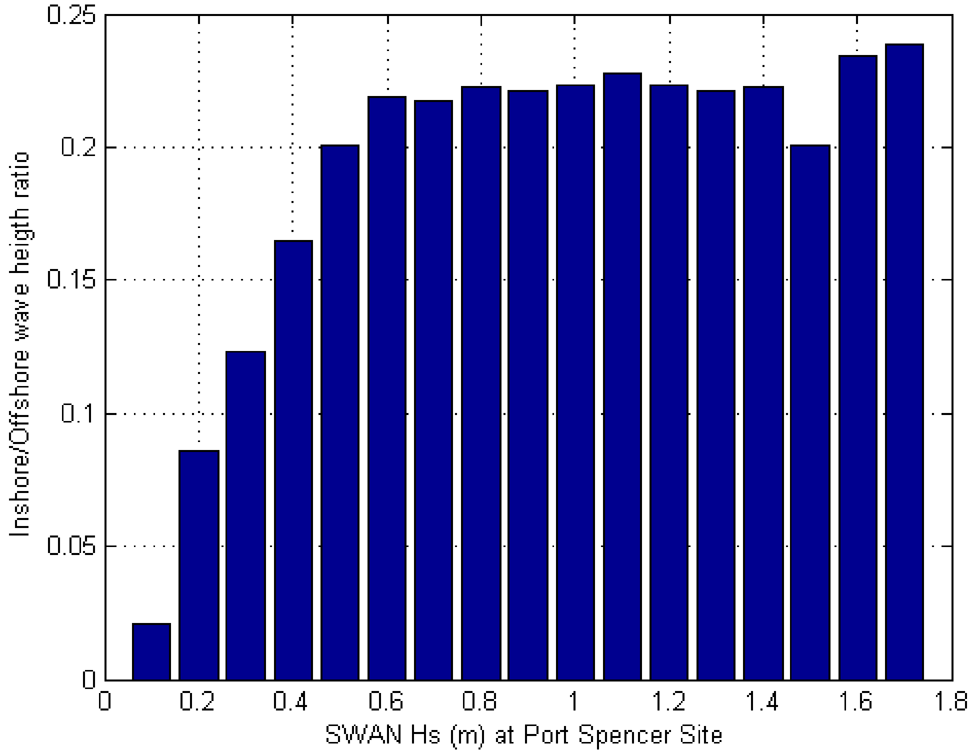

Figure 3.1. Comparison of modelled significant wave heights (Hs) at the Port Spencer site and the ratio of open-ocean MDI node (36° S 135° E) Hs and Port Spencer Hs for the inclusive period February 2005 to July 2011.

Table 3.1. Directional and non-directional percentage annual exceedance probabilities (%AEP) for significant wave height (Hs) based on 14 year GRB1 MDI record (1997-2010) for node in open-ocean offshore of Spencer Gulf (36°S 135°E).

Return Period (years)	% AEP	Significant Wave Height (Hs m)								
		Direction (45° centred binning)								Non Directional
		N	NE	E	SE	S	SW	W	NW	
1	100	4.1	3.0	2.3	4.3	5.6	9.3	8.6	5.5	9.5
5	20	4.9	3.7	2.6	5.0	6.6	10.4	10.0	6.3	10.6
10	10	5.2	3.9	2.7	5.3	7.0	10.8	10.6	6.7	11.0
25	4	5.6	4.3	2.9	5.6	7.6	11.4	11.3	7.1	11.6
50	2	5.9	4. 6	3.0	5.9	7.9	11.9	11.9	7.5	12.1
100	1	6.2	4.8	3.1	6.2	8.3	12.3	12.3	7.8	12.5

3.2 Locally Generated Seas within Spencer Gulf

As part of the previous work undertaken by ASR, a wave generation model was developed to predict waves within Spencer Gulf based on local winds. The model was driven by wind records from Port Lincoln Airport and showed reasonable calibration with locally generated sea waves recorded at the site (i.e.: wave conditions with periods less than 6 s given the available fetch within Spencer Gulf). Model calibration against deployment data was improved by scaling the Port Lincoln wind speeds upward by a factor of 1.1.

In order to predict extreme sea waves generated within Spencer Gulf, the long term wind record from Port Lincoln (including the upward scaling of wind speeds) was used to determine extreme winds which were in turn used to simulate waves using the WGEN model (ASR 3DD Suite 2011). Wind records for Port Lincoln from 1992 to 2010 were assessed and directionally binned (Table 3.2). WGEN was run for these conditions and the predicted significant wave heights at the Project for the various annual exceedance probabilities and directions are shown in Table 3.3. The largest predicted significant wave heights from locally generated seas within the gulf originated from the southeast, with heights of 1.8 m to 2.2 m (Tp = 6 s) for return periods from 1 to 100 years, respectively.

Table 3.2. Port Lincoln directional and non-directional annual exceedance probabilities (%AEP) for wind speed (ms^{-1}) based on 18 year record (April 1992 to Feb 2010).

Return Period (years)	% AEP	Wind Speed (ms^{-1})								
		Direction (45° centred binning)								Non Directional
		N	NE	E	SE	S	SW	W	NW	
1	100	21.6	12.2	12.1	14.0	14.8	16.6	20.9	21.6	18.9
5	20	24.7	13.9	13.3	15.3	16.1	18.6	23.3	24.4	20.3
10	10	26.0	14.5	13.8	15.8	16.6	19.4	24.3	25.5	20.9
25	4	27.7	15.4	14.4	16.5	17.3	20.5	25.5	27.0	21.7
50	2	28.9	16.1	14.9	16.9	17.8	21.2	26.4	28.0	22.3
100	1	30.1	16.7	15.3	17.4	18.3	22.0	27.3	29.1	22.8

Table 3.3. Predicted significant wave heights at the Port Spencer site from locally generated seas within Spencer Gulf (not including open-ocean swell penetration) based on directional wind speeds shown in Table 3.2. Prediction based on output from WGEN wave generation modelling.

Return Period (years)	% AEP	Significant Wave Height (Hs m)								
		Direction (45° centred binning)								Non Directional
		N	NE	E	SE	S	SW	W	NW	
1	100	1.4	1.3	1.5	1.8	1.7	1.3	0.7	0.7	n/a
5	20	1.5	1.5	1.7	1.9	1.8	1.4	0.8	0. 8	n/a
10	10	1.6	1. 6	1.8	2.0	1.9	1.5	0.8	0.8	n/a
25	4	1.7	1.7	1.8	2.1	1.9	1.6	0.9	0.9	n/a
50	2	1.8	1.7	1.9	2.1	2.0	1.6	0.9	0.9	n/a
100	1	1.9	1.8	2.0	2.2	2.1	1.7	0.9	0.9	n/a

Note: Peak periods range 3 - 6s.

3.3 Summary

Extreme waves from open-ocean swell penetration and locally generated seas within Spencer Gulf have been addressed separately. Analysis of 14 years of open-ocean swell wave data and SWAN modelling indicates that significant wave heights of 2.05 m to 2.71 m (1 to 100 year return period, respectively) may be experienced at the Project site from open-ocean swell penetration.

Analysis of 18 years of local wind data and WGEN wave generation modelling within Spencer Gulf indicates significant wave heights of 1.80 m to 2.20 m (1 to 100 year return period respectively) may be experienced at the Project site from locally generated seas (Tp < 6 s).

Suresh et al (2010) estimated that wind waves contributed 40% to the total Hs at a site on the east coast of India during the monsoon. The remaining 60% of the Hs is made up swell generated waves of higher period. For this open coast site the resulting sea-state consists of a bimodal frequency distribution. The above analysis shows that during periods of large ocean swells and stronger onshore winds 55% of the Hs at Port Spencer would be due to ocean swells and 45% would be due to wind generated waves (shorter periods) with the total wave heights as shown in Table 3.4.

Table 3.4. Extreme wave analysis return periods and percentage annual exceedance probabilities (%AEP) for the Port Spencer site. Wave heights are made up of return period waves for swell and wind generated waves.

Return Period (y)	% AEP	Wave Height (Hs)
1	100	3.7
5	20	4.0
10	10	4.2
25	4	4.4
50	2	4.5
100	1	4.7

4 POTENTIAL CHANGES TO FLOWS

In this section of the report a summary of the potential changes in wave induced flows is presented.

Both summer and winter wave climates have been modelled to quantify the potential change in wave induced currents and beach response due to the presence of a vessel at the jetty. Results for both wave conditions are similar (Figure 4.1 to Figure 4.4). Wave heights directly in the lee of a vessel are reduced by an average of 0.4 to 0.7 m. Wave heights directly inshore of a vessel would be reduced by around 0.5 m and this would be offset by a slight increase in wave height to the north and south of the jetty. The modelled hydrodynamic impact due to the changes in wave height caused by the presence of a vessel shows two converging circulation cells in the lee of the jetty. The maximum change in wave induced currents modelled is less than 0.15 ms^{-1}. Maximum changes would occur at either side of the headland directly inshore of the jetty.

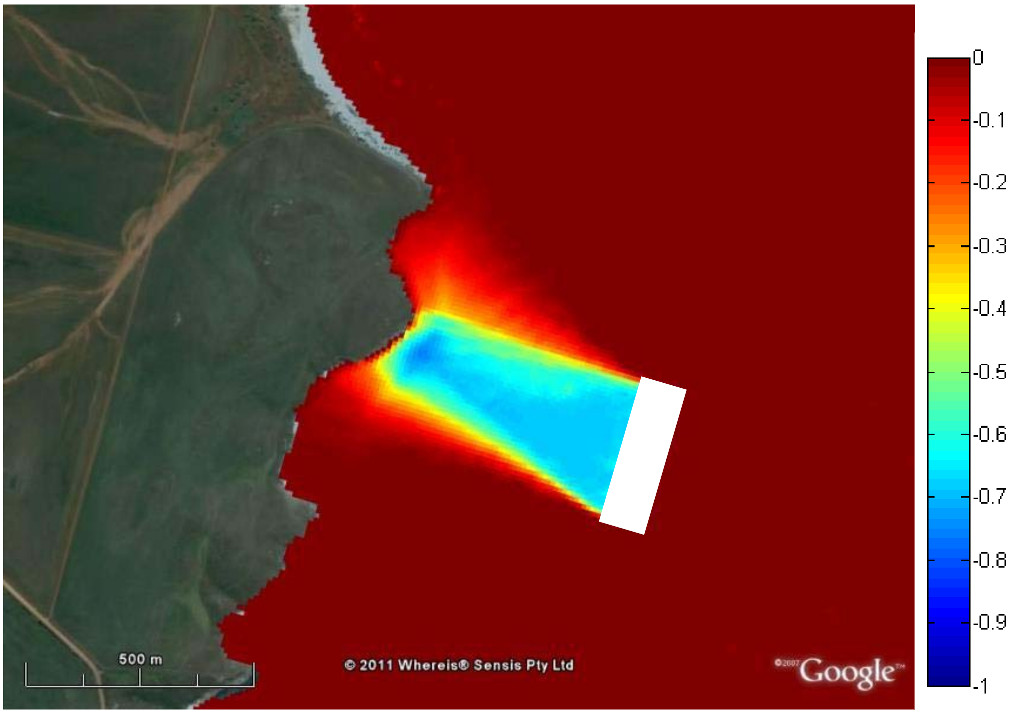

Figure 4.1. Predicted difference in wave height (m) due to moored vessel for summer conditions. White rectangle represents vessel.

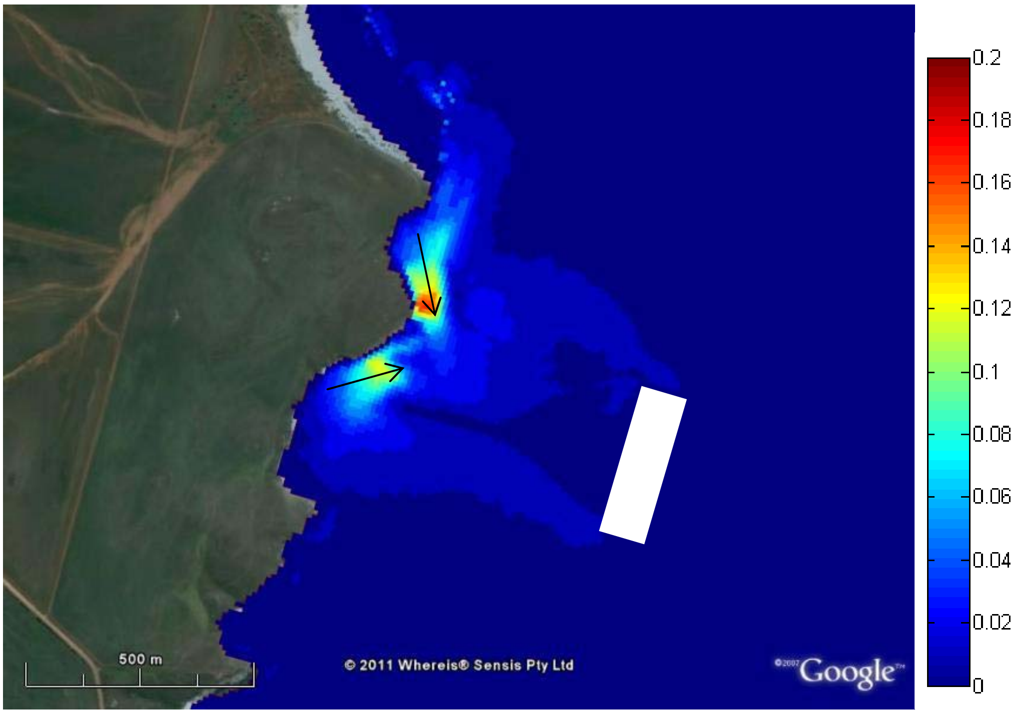

Figure 4.2. Difference in wave induced currents (ms^{-1}) due to the Project with moored vessel for summer conditions. White rectangle represents vessel, black arrows show direction of wave induced currents.

Figure 4.3. Predicted difference in wave height (m) due to moored vessel for winter conditions. White rectangle represents vessel.

Figure 4.4. Predicted difference in wave height (m) due to moored vessel for winter conditions. White rectangle represents vessel.

5 SEDIMENT TRANSPORT MODELLING

In this section of the report, the existing sediment transport regime at the Project is quantified and the likely changes to sediment are transport identified. The sediment transport modelling was examined using a three stage process as follows:

1. Modelling wave conditions
2. Modelling wind and tidally driven hydrodynamics
3. Combining the results of these models to investigate the effects and combined wave and hydrodynamically driven sediment transport.

The models were run with existing conditions (pre-construction) and with a vessel at the jetty to investigate the effect of the development on the potential for sediment mobility throughout the model domain. The focus of this component of work is to quantify the sediment transport regime in and around the jetty structure rather than sediment transport in the near shore zone (which has been modelled using 2DBEACH – see Section 6).

5.1 Wave Modelling

Wave modelling was carried out using 2DBEACH (ASR 3DD Suite 2011) for the summer and winter wave conditions (Figure 5.1 and Figure 5.2) although the previous modelling showed little difference between the two wave conditions. This modelling used the results presented in Grant et al. (2010a) and is summarised in Section 6.

5.2 Hydrodynamic Modelling

The hydrodynamic model was created using MODEL 3DD (ASR 3DD Suite 2011) and driven using the observed winds and currents at the Project. A boundary generation technique known as the 'Body Force Method' was adopted. The body force is a surrogate for a calculated sea gradient, obtained by inverting the vertically-averaged momentum equation and solving it using measurements of currents, sea levels and winds. In the simplest terms, the unmeasured pressure gradient force that must have been present in the ocean that was responsible for creating the observed oscillations in the measured currents is found.

The hydrodynamic model runs calibrate well for current speed and direction and for sea level for the summer and winter runs (Figure 5.3 and Figure 5.4). Deviations from the observed currents are driven primarily by bathymetric gradients. The calibrated hydrodynamic model was used as a basis for sediment transport modelling together with the wave model.

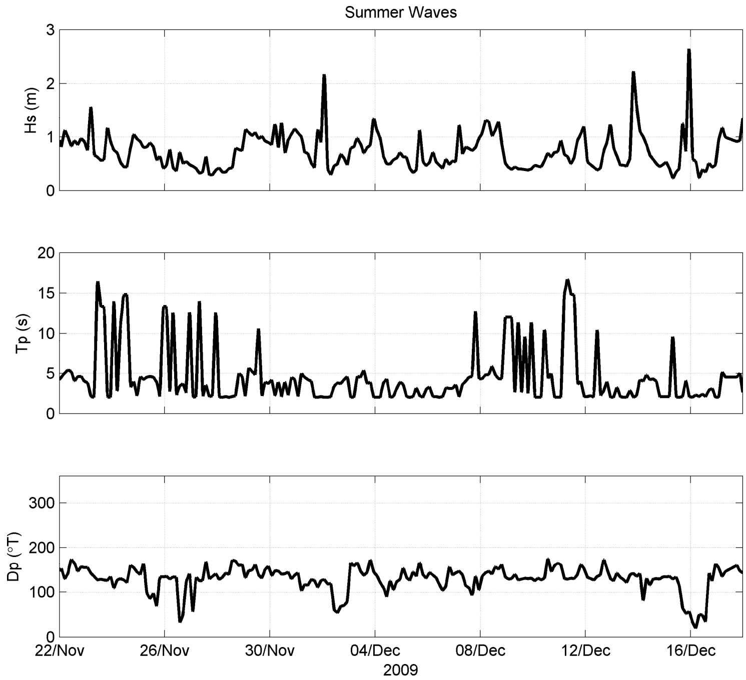

Figure 5.1. Summer wave record derived from winter field data collection programme showing significant wave height (Top), peak period (middle) and peak direction (bottom).

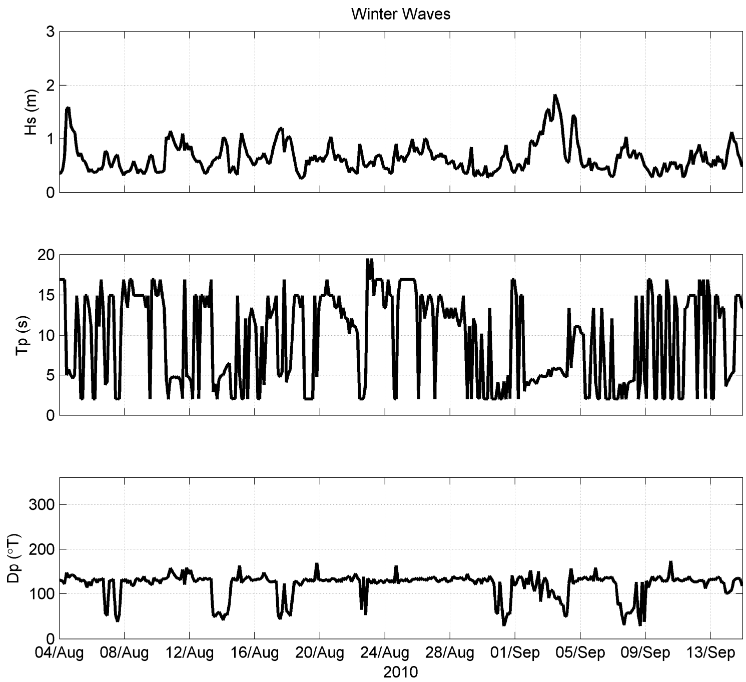

Figure 5.2. Winter wave record derived from winter field data collection programme showing significant wave height (Top), peak period (middle) and peak direction (bottom).

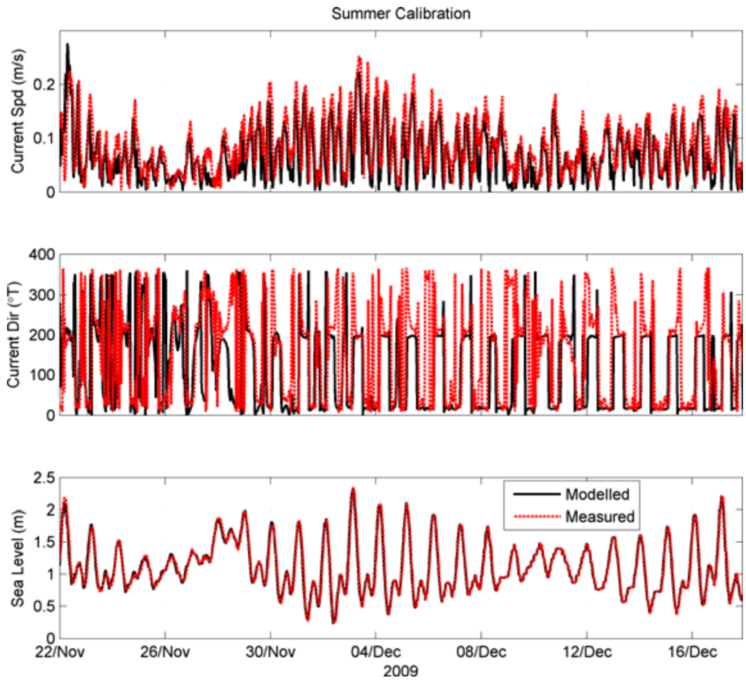

Figure 5.3. Summer calibration of current speed (top), current direction (middle) and sea level for hydrodynamic modelling of winter conditions.

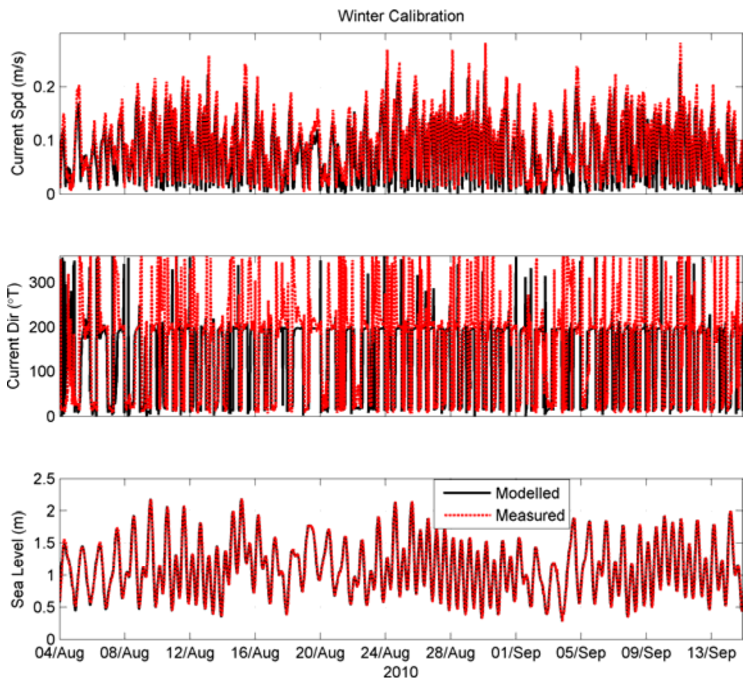

Figure 5.4. Winter calibration of current speed (top), current direction (middle) and sea level for hydrodynamic modelling of winter conditions.

5.3 Sediment Transport Modelling

Sediment transport throughout the site is expected to be influenced by a combination of wave and hydrodynamic influences. Both the current speed and the wave heights are by themselves moderate with current speeds of approximately 0.25 ms^{-1} at peak flow during spring tides and observed wave heights that are occasionally of the order of 2 m Hs. Tidal currents by themselves are not strong enough to suspend and transport sediment in large volumes, but become a major transporter of sediments which have become suspended by waves.

The hydrodynamic and wave models were combined to provide an overview of the interaction of the waves and tidal/wind driven currents using a methodology outlined in Soulsby (1997). The full methodology is described in Appendix 1. Rather than explicitly modelling the bed level changes over the site we used an approach that yields the volumetric transport rate which is the volume (m^3) of sand moving per metre per unit time (s). This allows an investigation into the degree to which sediment will be mobilised over the model domain without examining pathways of sediment movement. Simulating the same period both with and without a vessel present in the model shows how bed erodibility will change in the presence of a vessel. Results were taken from a three week model run but were scaled to give annual rates of transport. The results are split into two categories: gross transport rates, which is the sum of the absolute quantities of sand that are suspended in each cell; and the net transport rates, which are the vector averaged quantities of transported sediment in each cell. Given the similarities in the winter and summer wave climates shown in the wave and hydrodynamic modelling, only the winter wave climate is presented here. The results are shown in Figure 5.5 to Figure 5.10 and show gross and net transport rates with and without a vessel present as well as difference plots of gross and net transport rates with and without a vessel present. In all cases the transport rates near the shoreline are greatest where bed orbital velocities are at their largest. However for net transport rates nearshore values were set to zero to improve the visualisation of net transport in the lee of a vessel.

For both net and gross transport rates the values are consistent outside the breaker zone in the absence of a vessel, with a gross rate of approximately 350 m^3/annum/m and net transport of approximately 50 m^3/annum/m. However in the presence of a vessel, both gross and net transport rates drop to nearly zero in its lee, where very little wave energy penetrates. This is offset by an increase in net transport rates to the south of the jetty structure of around 70 m^3/annum/m. With an occupancy rate of 20%, the change in net transport rates would be of the order of 10-15 m^3/annum/m. Assuming this decrease in transport rate leads to deposition along the length of the berthing jetty it could be expected that the bed level change in the lee of the vessel could range from 0.03 to 0.05 m/annum. It is recommended that regular surveys of the area inshore of the jetty are carried out at intervals to monitor bed level changes as described in more detail in Section 6.

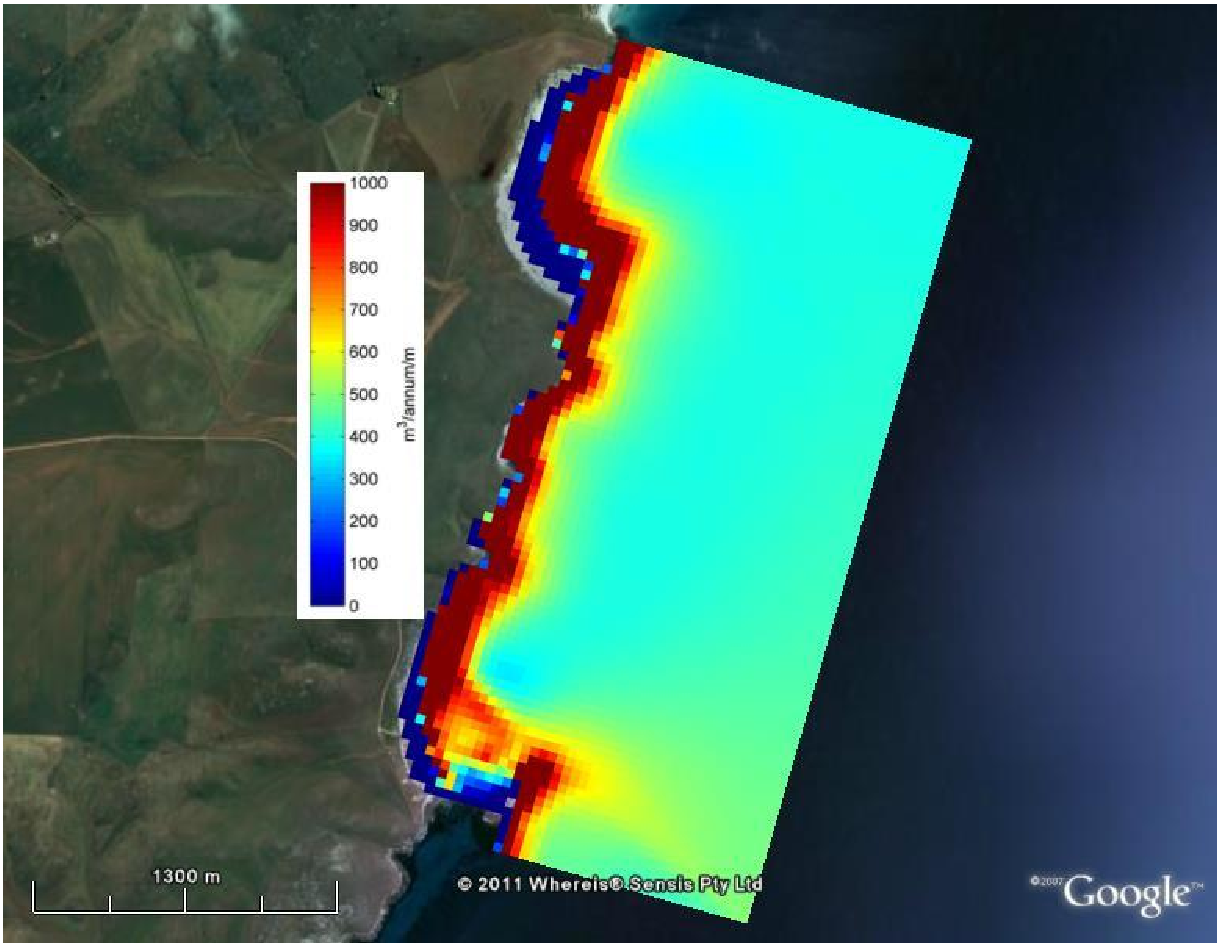

Figure 5.5. Gross sediment flux rates (m^3/annum/m) across the model domain for winter wave and tidal conditions for existing conditions.

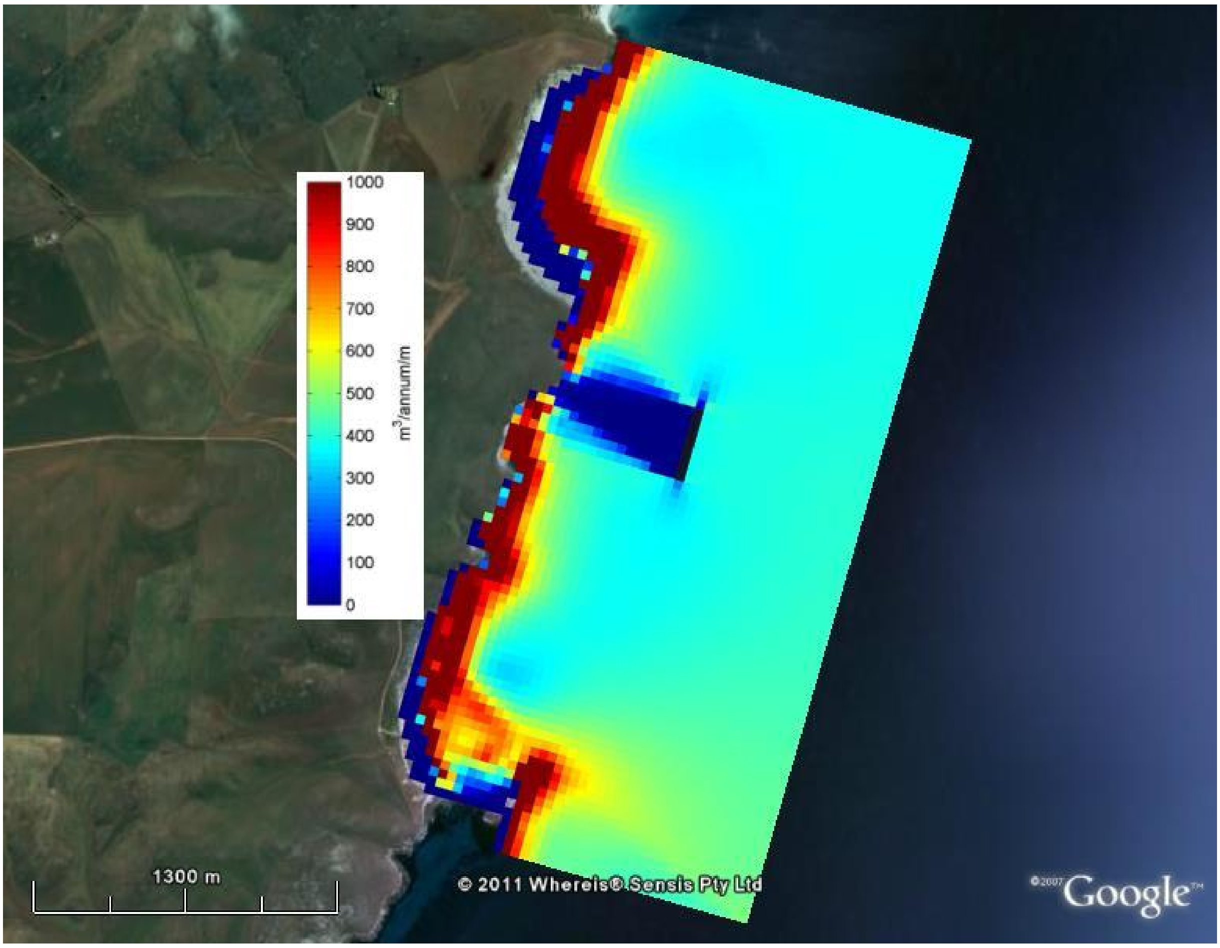

Figure 5.6. Gross sediment flux rates (m^3/annum/m) across the model domain for winter wave and tidal conditions with the presence of a vessel at the jetty.

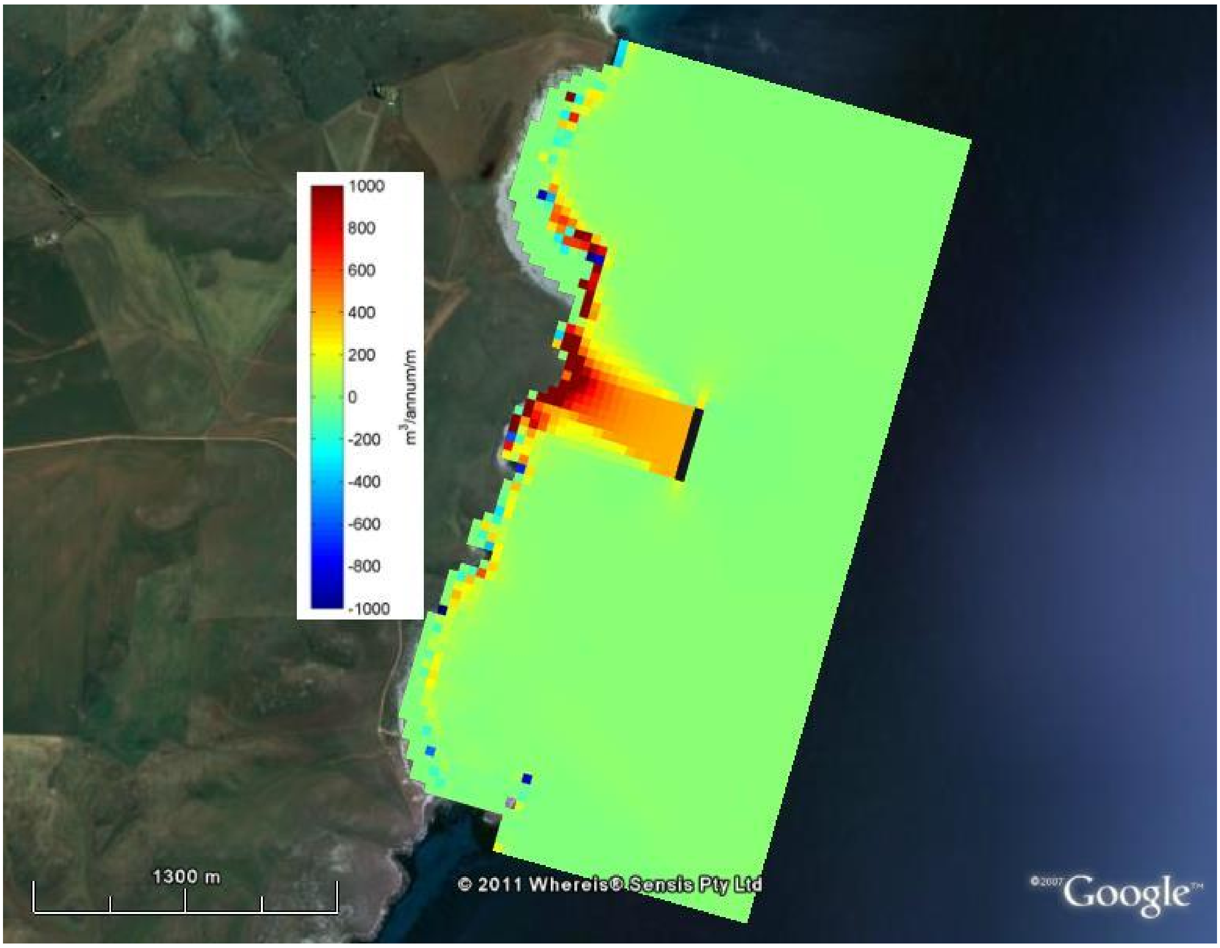

Figure 5.7. Difference in gross sediment flux rates (m^3/annum/m) due to the presence of a vessel at the jetty.

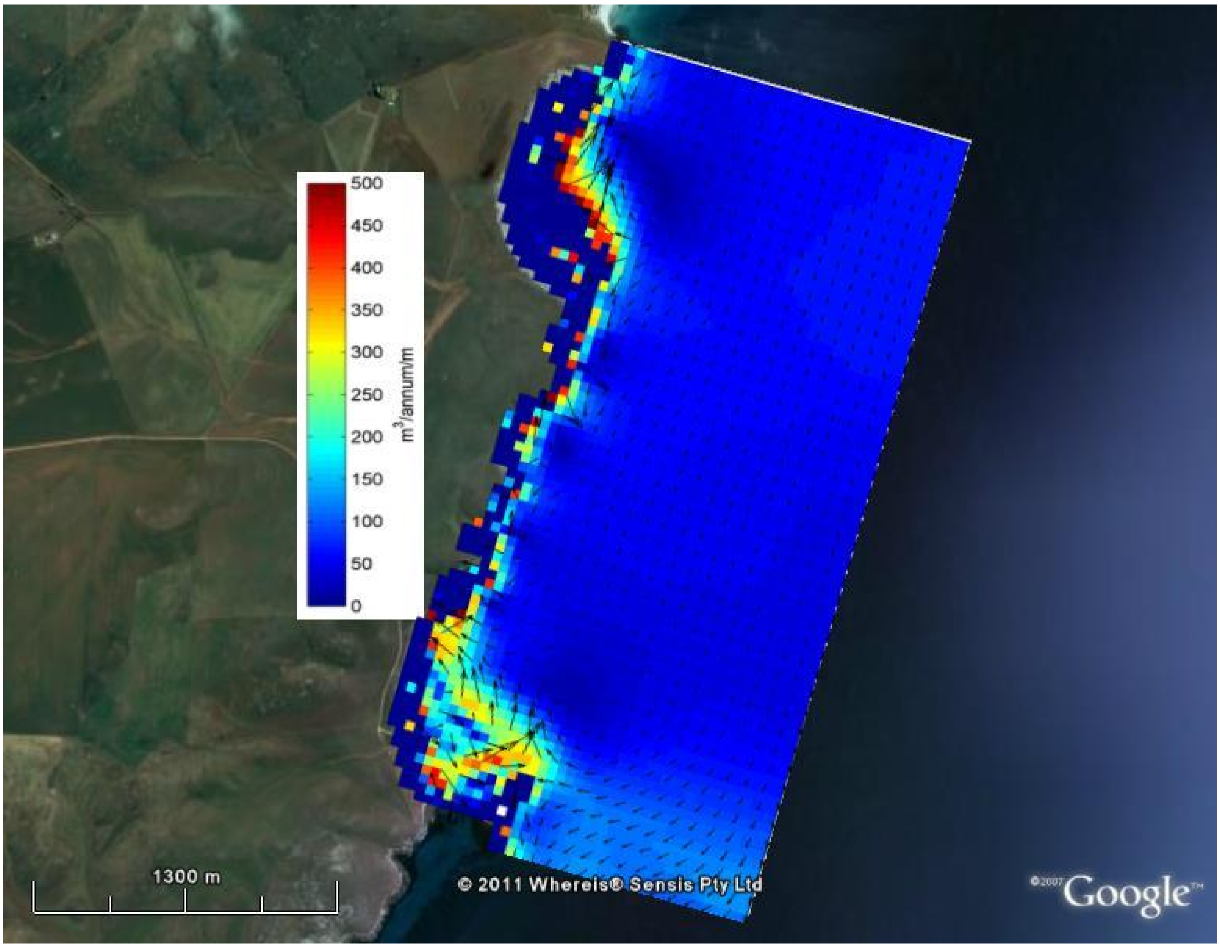

Figure 5.8. Net sediment flux rates (m^3/annum/m) across the model domain for winter wave and tidal conditions for existing conditions.

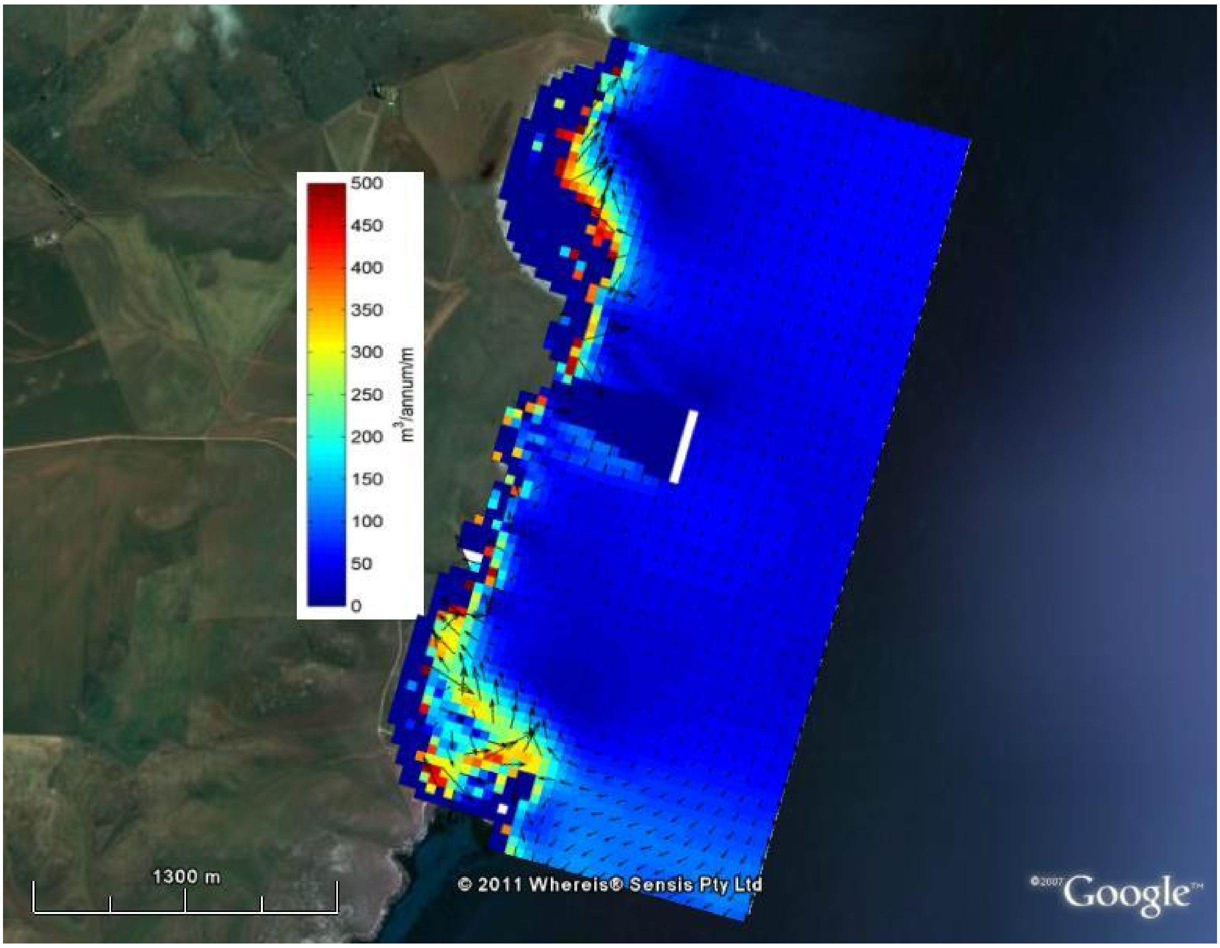

Figure 5.9. Net sediment flux rates (m^3/annum/m) across the model domain for winter wave and tidal conditions with the presence of a vessel at the jetty.

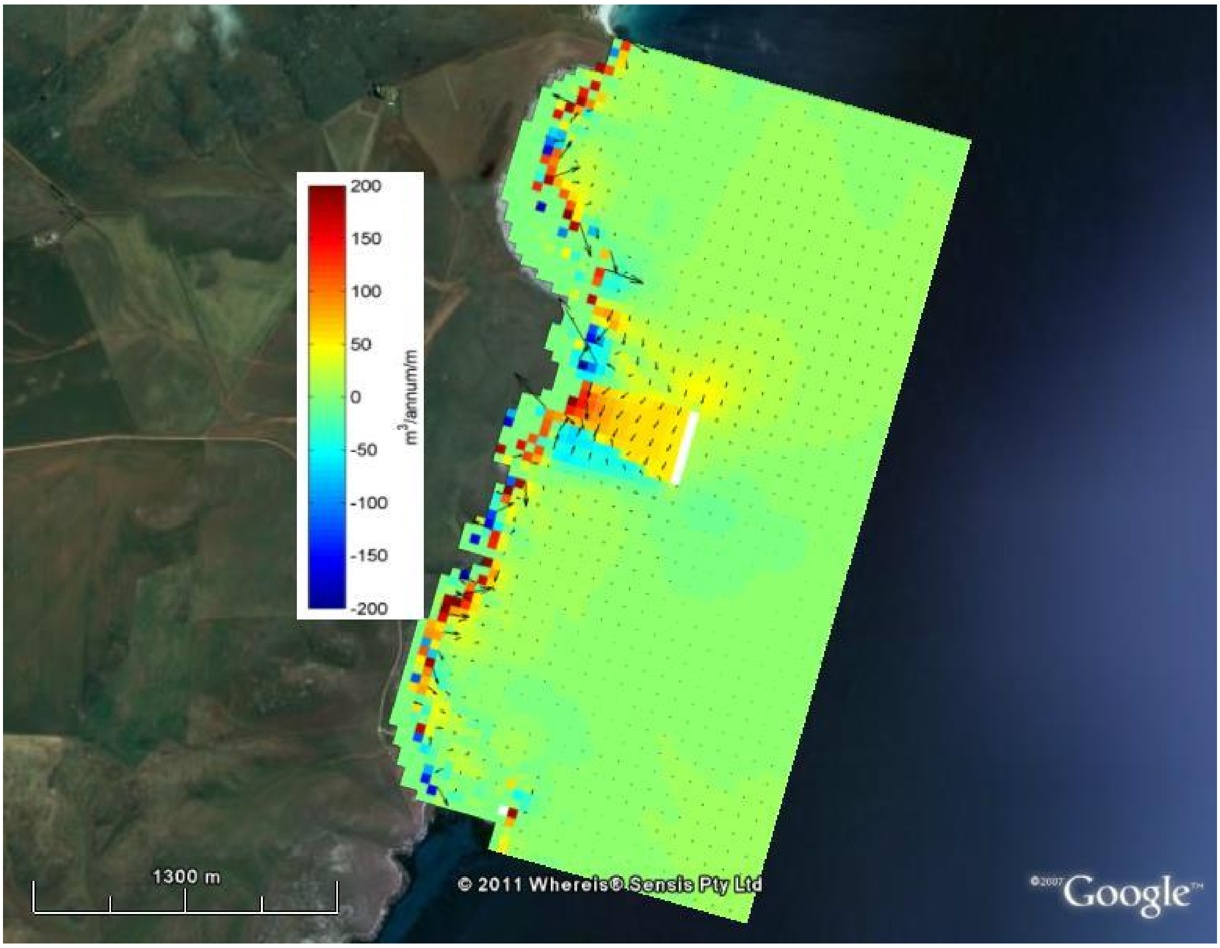

Figure 5.10. Difference in net sediment flux rates (m^3/annum/m) due to the presence of a vessel at the jetty.

6 BEACH RESPONSE

As discussed above the presence of a vessel at the jetty will result in changes to wave heights directly inshore of it (offset by a slight increase in wave height to the north and south of the Project). These changes result in changes to flows in the near shore zone, which results in an altered beach response. Earlier modelling (Grant et al. 2010a) predicted that bed level changes immediately inshore of the jetty may change by less than 0.1 m/annum (compared to the predicted bed level changes of 0.03 to 0.05 m/annum). An increase in sediment is predicted to build up immediately inshore of the jetty. North of the jetty accretion may be slightly reduced and the area where erosion is currently predicted to be occurring may decrease. These results assumed that a vessel would be present at the site all of the time.

It is predicted that a vessel will be moored at the jetty for only 20% of the time (see Section 2.6). For this occupancy time (over a 50 year time span) the change in beach response (over the beach response that would occur under existing conditions) is shown in Figure 6.1. Areas of rocky reef are shown as solid black lines. For these areas no change to beach profiles is predicted to occur. In areas where there are no rocky reefs the predicted changes in bed levels will translate into a shoreward or seaward movement of the existing beach profile (depending on the existing beach slope). The area potentially impacted by the Project would be the beach to the north.

A regular programme of beach monitoring should be put in place prior to any site construction. It is recommended that six regularly spaced beach profiles are established along the length of this beach, with surveys to low water carried out at monthly intervals. Data collected over the next twelve months would give a good indication of the existing beach dynamics. In order to monitor changes post construction, quarterly beach profile surveys should be undertaken for at least 3 years. This data could be put in context of longer time frames by an analysis of any historical aerial photographs of the area. If mitigation is required sediment by-passing or renourishment option could be investigated.

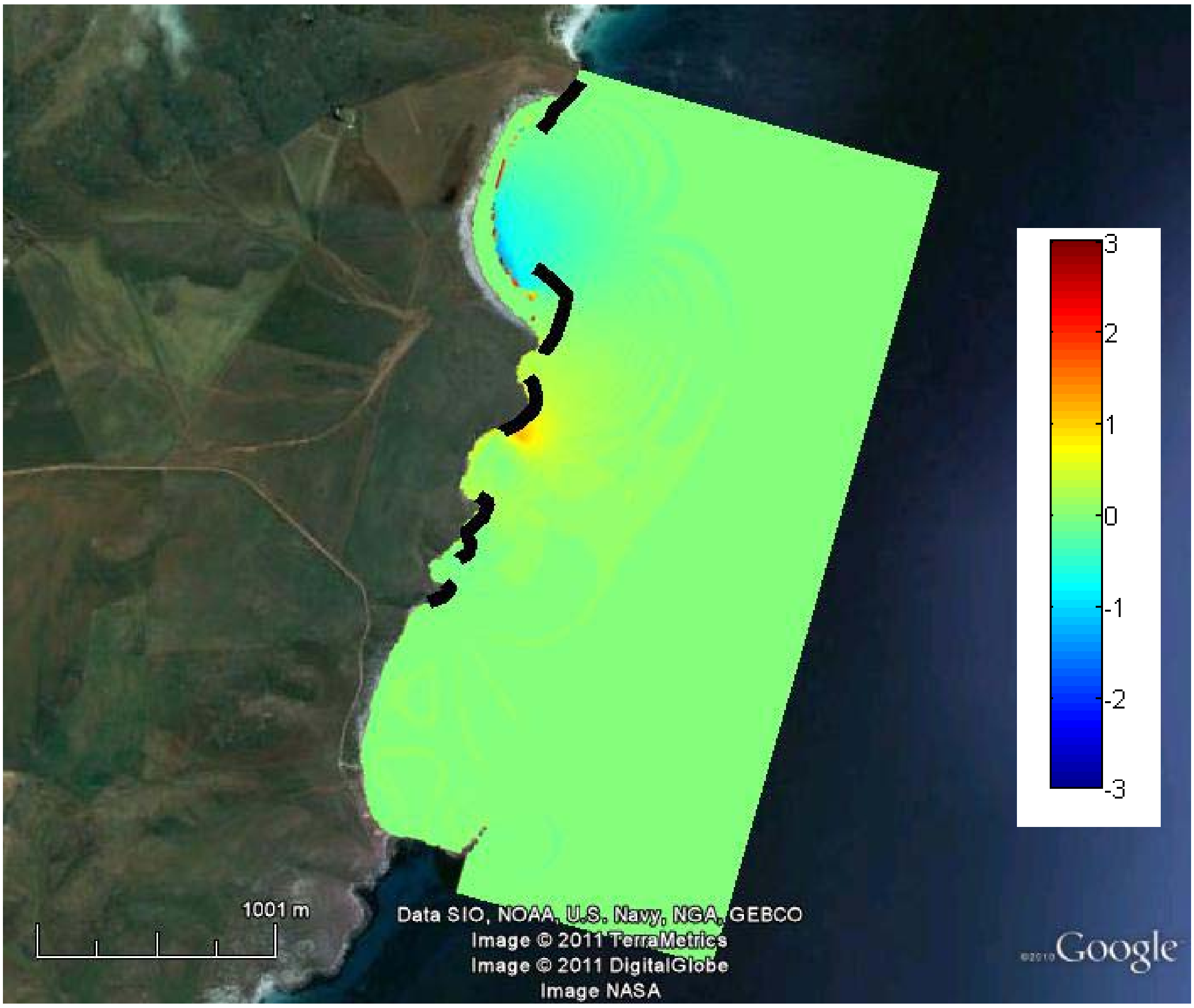

Figure 6.1. Predicted difference in bed level compared to existing conditions (over 50 years with moored vessel for 20% of the time). Black lines show locations of rocky headlands where predicted bed elevation changes will not occur but sediment transport through region will increase/decrease.

7 PILE EFFECTS

7.1 Scour Hole Dimensions

Scour holes are predicted to form in the immediate vicinity of individual piles on the jetty. The dimensions of these scour holes have been estimated from a range of empirical methodologies and the shape of the scour holes have been quantified through the use of a high resolution 2DBEACH hydrodynamic model (ASR 3DD Suite 2011). The predicted depth of the scour holes would range from 0.3 to 1.4 m. The long shore length of the scour holes is likely to be 0.6 to 2.0 m long with significantly lower cross shore dimension due to small cross shore currents that occur at the site. Importantly this analysis shows that there will not be any interference or accumulative effects of pile scour holes at the Project. Because of the keel clearance (i.e., minimum 2 m), the pile spacing and the relatively localised effects on flows, the formation of scour holes are unlikely to have any detrimental effects on vessel movement or manoeuvrability.

During the pile driving process, pile fabric filtering will be used around each pile so that turbidity effects will be minimal. Therefore no modelling of sediment plumes from the construction phase has been carried out.

7.2 Wave Transformation due to Scour Holes

Section 4 outlines the potential effects of the vessel on wave driven and tidally driven currents. In this section wave transformation effects of the scour holes are quantified in terms of the same summer and winter wave conditions modelled for the 2DBEACH simulations. The following figures show the assumed bathymetry with scour holes of 0.9 m depth (Figure 7.1), predicted mean wave heights (Figure 7.2) and predicted bed orbital velocities (Figure 7.3) which relate directly to potential for sediment transport. The plots show that there is virtually no difference in the wave transformation processes due to the scour holes with only small differences in predicted mean wave height and bed orbital velocities close to the shoreline (Figure 7.4). Essentially the potential increase in depth around each pile due to scour will have minimal impact on wave transformation processes and therefore no effect on beach dynamics.

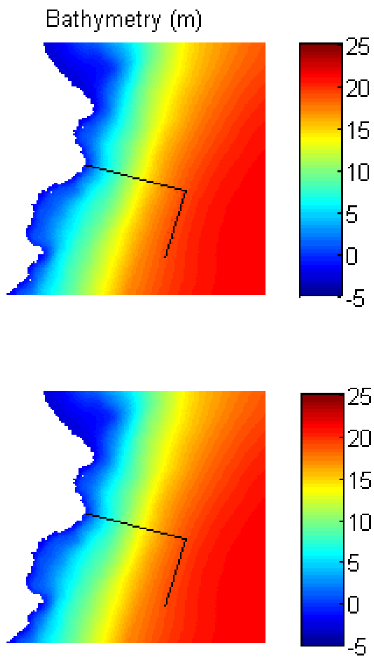

Figure 7.1. Bathymetry with 0.9 m scour holes (top panel) and existing bathymetry (bottom panel).

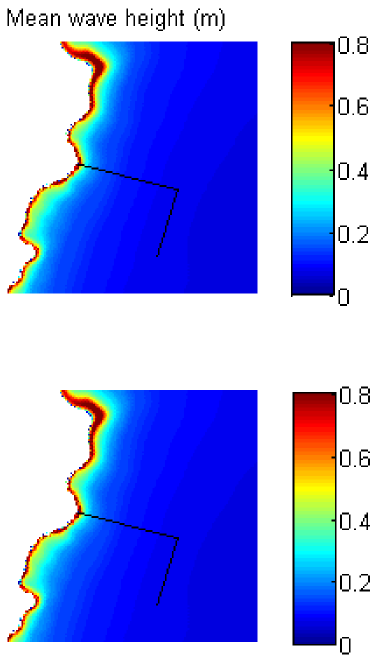

Figure 7.2. Predicted mean wave height (m) for summer wave conditions. Top panel shows results with 0.9 m deep scour bottom panel shows results for the existing bathymetry.

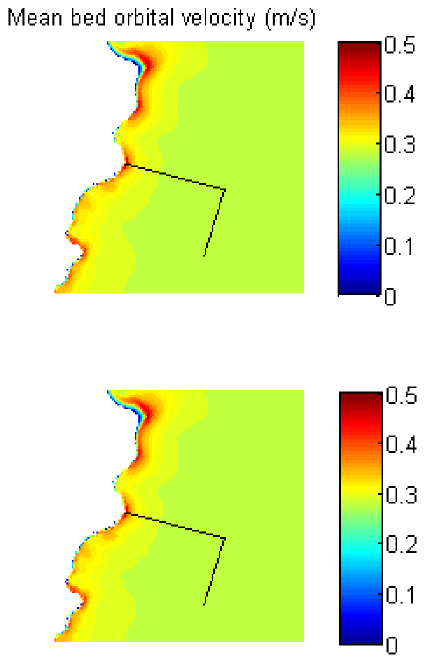

Figure 7.3. Predicted mean bed orbital velocity for summer wave conditions. Top panel shows results with 0.9 m deep scour bottom panel shows results for the existing bathymetry.

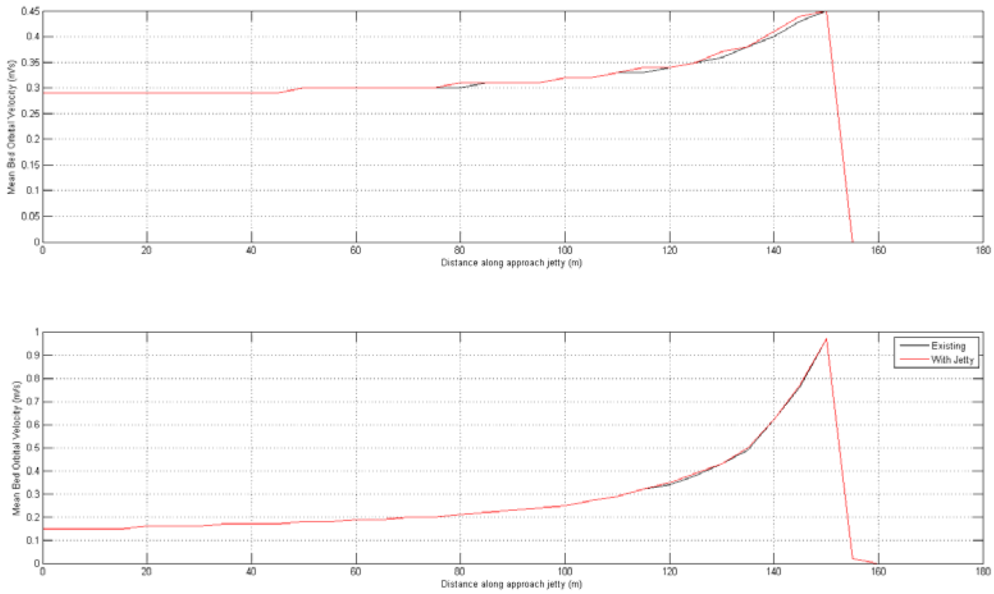

Figure 7.4. Results from wave transformation model for a transect along the length of the approach jetty. Top panel shows mean summer bed orbital velocity, bottom panel shows mean summer wave height.

8 CONCLUSIONS

This report has detailed the modelling and assessment of environmental conditions undertaken for the Port Spencer development including the following:

- extreme wind, wave and tidally driven currents at the Project
- quantification of existing flow regime
- existing sediment transport regime at the Project
- scour effects due to jetty and associated piling
- changes in flows due to the proposed development
- how the Project may alter sediment transport
- potential changes to beach profiles.

Extreme winds at Port Spencer are estimated at 22 to 30 ms^{-1} for return periods of 1 to 100 years, respectively. Extreme significant wave heights are estimated at 3.7 to 4.7 m for return periods of 1 to 100 years, respectively. Water level fluctuations measured during the deployments resulted from the tide, with smaller water level changes associated with wind and wave induced set up, barometric pressure effects and coastal trapped waves. The overall tidal range is on the order of 2.1 m while the other factors combined are on the order of 0.4 m.

Potential changes to the flow regime are driven primarily by changes in local wave heights associated with the presence of moored vessels. Modelled wave heights directly inshore of the vessel are shown to be reduced, offset by a slight increase to the north and south of the jetty. The modelled hydrodynamic impact due to the changes in wave height caused by the presence of a vessel shows two converging circulation cells in the lee of the jetty. Maximum changes in wave induced currents are less than 0.15 ms^{-1} predicted either side of the headland directly inshore of the jetty.

Sediment transport throughout the Port Spencer site is expected to be influenced by a combination of wave and hydrodynamic influences. Tidal currents by themselves are not strong enough to suspend and transport sediment in large volumes, but become a major transporter of sediments which have become suspended by waves. Subsequently as modelled the presence of a vessel reduces both net and gross transport rates in its lee given the reduction in wave energy. This is offset by an increase in modelled net transport rates to the south of the jetty. Considering vessel occupancy at a rate of 20% per annum, the change in net transport rates would be of the order of 10-15 m^3/annum/m. Assuming this decrease in transport rate leads to deposition along the length of the berthing jetty it could be expected that the bed level change in the lee of the vessel mooring could range from 0.03 to 0.05 m/annum.

Beach response associated with the changes in sediment transport, are predicted to result in accretion immediately inshore of the jetty. North of the jetty accretion may be slightly reduced, which may result in an erosionary beach response.

Scour holes are predicted to form in the immediate vicinity of individual piles on the jetty. The dimensions of these scour holes have been estimated. The analysis shows that there will not be any interference or accumulative effects of pile scour holes at the Project. The potential increase in depth around each pile due to scour will also have minimal impact on wave transformation processes and therefore no effect on beach dynamics.

The environmental effects modelled are shown to be relatively localised in relation to the Project and do not extend significant distances up and down coast. For example, Lipson Island is located approximately 1.5 km south of the Project, and the modelling undertaken indicated no significant changes in the environmental regime at this location.

To provide more confidence in our assessment, a programme of monitoring should be put in place. It is recommended that bathymetric surveys are established at the Project annually to monitor bed changes in the lee of the jetty. A regular programme of beach monitoring should also be put in place prior to any site construction. It is recommended that regularly spaced beach profiles are established along the length of the adjacent beaches, with surveys to low water carried out at monthly intervals. Data collected over the next twelve months would give a good indication of the existing beach dynamics. In order to monitor changes post construction, quarterly beach profile surveys should be undertaken for at least 3 years. This data could be put in context of longer time frames by an analysis of any historical aerial photographs of the area. If mitigation is required sediment by-passing or renourishment options could then be investigated.

APPENDIX 1

The volumetric transport rate q_t is defined as:

$$q_t = A_s \overline{U} \left[\left(\overline{U} + \frac{0.018}{C_D} U_{rms}^2 \right)^{1/2} - \overline{U}_{cr} \right]^{2.4}$$

Where:

$\overline{U}$ is the depth averaged current speed (taken from the hydrodynamic model)

C_D is the drag coefficient

$\overline{U}_{cr}$ is the critical velocity threshold for sediment suspension

U_{rms}^2 is the root-mean-squared wave orbital velocity (derived from the wave model)

A_s is a coefficient defined below

The value for $\overline{U}_{cr}$ is defined as

$$\overline{U}_{cr} = 0.19(d_{50})^{0.1} log_{10} \left(\frac{4h}{d_{90}} \right)$$

Where values for d_{50} and d_{90} (the 50th and 90th percentile grain sizes) were derived from sediment analysis undertaken by Golder Associates (Particle Size Distribution & Consistency Limits Test Reports No. 107661001).

$$T_z = 0.781 T_p$$

T_z is the zero up-crossing wave period and T_p is the peak wave period.

The coefficient has contributions from a bed level term (A_{sb}) and a suspended sediment term (A_{ss}) such that:

$$A_s = A_{ss} + A_{sb}$$

where

$$A_{sb} = \frac{0.005h(d_{50}/h)^{1.2}}{[(s-1)gd_{50}]^{1.2}}$$

$$A_{ss} = \frac{0.012 d_{50} D_*^{-0.6}}{[(s-1)gd_{50}]^{1.2}}$$

and

$$D_* = d_{50}\left[\frac{g(s-1)}{v^2}\right]^{1/3}$$

with

$$s = \frac{\rho_s}{\rho}$$

Finally the drag coefficient is calculated by the following identity.

$$C_D = \left[\frac{0.40}{ln(h/z_0) - 1}\right]^2$$

REFERENCES

Ansell, A. Gibson, R. And Barnes, M. (1997), Oceanography and marine biology, An Annual Review Volume 35.

Black, K.P. and Oldman, J.W. (1999). Wave mechanisms responsible for grain sorting and non-uniform ripple distribution across two moderate-energy, sandy continental shelves. Marine Geology Vol. 162, pp121-132.

Development Assessment Commission, South Australia (2011) Guidelines for the preparation of a Public Environmental Report Sheep Hill Deep Water Port Facility (Stage 1) on Eyre Peninsula.

Golder Associates (2010) Marine Geophysical Investigations at Sheep Hill near Tumby Bay, South Australia. Reference 107661001-006-L-Rev0.

Golder Associates (2011) Additional Marine Ecological Studies and Environmental Effects Assessment. Report Prepared for Centrex Metals Ltd. Reference 107661001-011-R-Rev0.

Grant, D., Greer, D., Cosotti, Y. (2010a) Lipson Cove Field Data Programme – Summary Report. ASR client report produced for Golder Associates.

Grant, D., Moores A. (2010b) Lipson Cove Field Data Report Deployment 5 – Winter Conditions. ASR client report produced for Golder Associates.

Moores, A., Lebreton, L., Oldman, J., Mead, S. Grant, D. (2010) Sheep Hill Ship Loader – Study of Morphological Effects. ASR client report produced for Golder Associates.

Soulsby, R. (1997). Dynamics of Marine Sands - a Manual of Practical Applications. ThomasTelford, 250 pp.

Suresh. R. R. V., Annapurnaiah, K., Reddy, K.G., Lakshmi, T.N., Balakrishnan Nair, T M. (2010) Wind Sea and Swell Characteristics off East Coast of India During Southwest Monsoon International Journal of Oceans and Oceanography Volume 4, 1 pp. 35-44.