Technical review of the Semaphore Park trial breakwater, South Australia

Author:
Carley, J.T.; Mariani, A.

Publication details:
Report No. UNSW Water Research Laboratory Technical Report No. 2007/14

Publication Date:
2007

DOI:
https://doi.org/10.4225/53/58e1b831d4b69

THE UNIVERSITY OF NEW SOUTH WALES

water research laboratory

Manly Vale N.S.W. Australia

TECHNICAL REVIEW OF THE SEMAPHORE PARK TRIAL BREAKWATER, SOUTH AUSTRALIA

by

J T Carley and A Mariani

Technical Report 2007/14

August 2007

THE UNIVERSITY OF NEW SOUTH WALES
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
WATER RESEARCH LABORATORY

TECHNICAL REVIEW OF THE SEMAPHORE PARK TRIAL BREAKWATER, SOUTH AUSTRALIA

WRL Technical Report 2007/14 August 2007

by

J T Carley and A Mariani

https://doi.org/10.4225/53/58e1b831d4b69

Water Research Laboratory
School of Civil and Environmental Engineering
University of New South Wales ABN 57 195 873 179
King Street
Manly Vale NSW 2093 Australia

Telephone: +61 (2) 9949 4488
Facsimile: +61 (2) 9949 4188

Technical Report No	2007/14
Report Status	Final
Date of Issue	August 2007
WRL Project No.	06111
Project Manager	James Carley

Title	Technical Review of the Semaphore Park Trial Breakwater, South Australia
Author(s)	James T Carley and Alessio Mariani
Client Name	Coastal Protection Branch, Department for Environment and Heritage South Australian Government
Client Address	GPO Box 1047 Adelaide SA 5001
Client Contact	James Guy
Client Reference	DEH 260/11/14

EXECUTIVE SUMMARY

ES.1 Reason for the Study

Adelaide's beaches have had a long history of erosion. This led to a trial breakwater being constructed from geotextile tubes at Semaphore Park to trap sand by forming a salient. The sand can be extracted and used for replenishment of eroding beaches to the south. For this project, sand carting commenced in September 2003 and the breakwater construction commenced in April 2004. The offshore breakwater has been monitored by regular surveys since construction. This report details analysis of the monitoring data and additional information to determine the performance of the trial structure.

ES.2 Measured versus Predicted Performance

The as-built crest level was +1 m AHD, which later reduced to -0.9 m AHD. When the crest level was inferred to be at the original level, the salient size and shape, and extent of updrift and downdrift effects were well approximated by the original modelling. Sustainable sand yields were found to be in the range 22,000 to 40,000 m^3/year.

The performance with the crest at -0.9 m AHD was also predicted in the original modelling, however, this reduced sustainable sand yields to a maximum of 9,000 m^3/year.

ES.3 Recommendations for Permanent Structure and Sustainable Sand Yields

The trial breakwater has largely performed as intended, so based on a technical assessment a permanent structure is warranted. The crest should be at the original intended design level of +1 m AHD. Based on preliminary design, the rock size needed to make the breakwater permanent would exceed the available space above the geotextile tube structure, and therefore the trial structure needs to be removed. This means that the permanent structure need not be located in the original Bower Road location.

If the permanent structure is located off Hart Street, this would reduce the downdrift erosion and the required sand carting distance and quantities from 10,000 m^3/year to 5,000 m^3/year, subject to ongoing monitoring. This would result in greater distance between the breakwater and the Semaphore Park erosion area to the south, which would result in higher costs if the sand was to be carted in trucks. However, for a sand pumping system, this distance is less than that needed for a booster station, so the additional pipe costs (between the salient and the Semaphore Park erosion zone) are more than offset by the reduced distance and sand carting quantities between Semaphore Jetty and downdrift of the salient (see below). Sustainable sand yields for harvesting from the salient and pumping the sand to the Semaphore Park erosion area to the south are estimated to be 40,000 m^3/year subject to ongoing monitoring, provided that the salient is harvested regularly so that it is maintained in a small state. Accretion volumes in the salient reduce as its size increases.

ES.4 Costs

A permanent breakwater designed using desktop techniques and comparison to other stable coastal structures around Adelaide is estimated to cost $1.2 Million subject to detailed design, which may require physical model testing.

The following three alternatives are compared below:

1. A field of five permanent breakwaters similar in size to the "as designed" Bower Road structure, with approximately 200 m gaps between each breakwater.
2. A single permanent breakwater located off Bower Road to act as a sand trap, together with sand pumping from the salient into the Semaphore Park erosion zone.
3. A single permanent breakwater located off Hart Street to act as a sand trap, together with sand pumping from the salient into the Semaphore Park erosion zone.

For all three options, replenishment of the downdrift area of the salient would still be undertaken by sand carting from near Semaphore Jetty, at a rate of 10,000 m^3/year for alternatives 1 and 2, and a rate of 5,000 m^3/year for alternative 3. Options 2 and 3 involve pumping sand (40,000 m^3/year) from the salient to the Semaphore Park erosion zone.

The net present costs at 7% discount rate over 20 years are:

1. Field of five breakwaters: $5.8 Million.
2. Single permanent breakwater at Bower Road: $2.3 Million.
3. Single permanent breakwater at Hart Street: $2.2 Million.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

This report details a technical review of the Semaphore Park trial breakwater in Adelaide, South Australia. The breakwater was constructed as a trial structure to trap sand which can be extracted and used for replenishment of eroding beaches to the south. The project has been implemented by the Coastal Protection Branch (CPB) of the Department for Environment and Heritage, South Australia.

Townsend (2005) reported the following construction sequence for the breakwater:
"Mobilisation began in September 2003........ Works were halted over the summer holidays because of high beach populations. [Sand] Carting resumed in February 2004 and breakwater construction began in earnest in April." In early 2005, one tube remained to be completed.

The offshore breakwater has been monitored by regular surveys since construction. This report details analysis of the monitoring data and additional information to determine the performance of the trial structure.

The Adelaide coast has had numerous studies undertaken. The principal works leading to the Semaphore Park trial breakwater are Townsend (1999), Coastal Engineering Solutions (2000) and Carley *et al*. (2001).

The work undertaken in Carley *et al*. (2001) was undertaken on an AMG84 geodetic grid. The work in this report was undertaken on a GDA94 grid in accordance with the supplied data. For the Semaphore Park region the following approximate corrections are needed to convert plan positions.

GDA94 North = AMG84 + 178.5 m
GDA94 East = AMG84 + 121.8 m.

Selected figures from Carley *et al*. (2001) have been reproduced in Appendix E to make this report self contained. Note that these Appendix E figures are presented in the AMG84 geodetic grid, and therefore need to be translated as described above to conform to the GDA94 grid used in the body of this report.

2. AS DESIGNED, AS BUILT AND PRESENT BREAKWATER

The as-designed breakwater sections are shown in Figures 2.1 (cross sections) and 2.2 (long section). It can be seen that the design crest level (following initial settlement) was 1.0 m AHD (Australian Height Datum). Also shown in Figure 2.2 is data from an as-built survey (York Civil, 21/12/2004), where it can be seen that the design crest level of +1 m AHD was broadly achieved.

The plan dimensions of the surveyed and modelled breakwater are shown in Table 2.1, where it can be seen that in the November 2006 survey the breakwater was slightly shorter in length than designed, but apparently wider (see below). The plan positions (corrected to GDA94) of the centre of the surveyed and modelled breakwater are shown in Table 2.2, where it can be seen that in the November 2006 survey the centre of the breakwater was located within 6 m of its planned position.

Table 2.1
Plan Dimensions of Breakwater

	November 2006 Survey		**Alternative design**		**GENESIS modelled design**	
	length (m)	width (m)	length (m)	width (m)	length (m)	width (m)
At base	196.9	12.9	208.4	8.7	209.1	17.6
At crest	180.3	5.9	200.6	4.4	200.6	6.6

Table 2.2
Centre of Breakwater

Centre of structure (m GDA94)	**November 2006 Survey**	**Alternative design**	**GENESIS modelled design**
East	268,974.1	268,971.8	268,971.8
North	6,140,690.0	6,140,686.3	6,140,686.3

The most recent structure survey (November 2006) shows that the crest level was generally in the range -1 to 0 m AHD, with an average level of -0.9 m AHD (Figure 2.2). CPB staff reported that this lowered crest is due to the top row of tubes rolling off. Settlement of the structure also cannot be excluded. The increased width at the base (November 2006 survey versus alternative design) shown in Table 2.1 is due to rolled off geotubes. The precise date or time period of this change in crest level from +1 to -0.9 m AHD is unknown, other than that it occurred some time between 21/12/2004 and November 2006. From the analyses undertaken in Section 4, the lowering has been inferred to have occurred in approximately March 2006.

A lowering of the crest from +1 to -0.9 m AHD was estimated to have the effect of increasing the average wave transmission coefficient for sea waves from 0.2 to 0.8 (Carley *et al*. 2001, Figure 6.3 reproduced in Appendix E of this report). Based on GENESIS "scoping" stage modelling undertaken in Carley *et al*. (2001), this would reduce the net effect of the 10 year salient width from approximately 120 m to 25 m (Carley *et al*. 2001, Figure 8.16 reproduced in Appendix E of this report).

A detailed analysis of storm wave events and the wave climate during the monitoring period was not part of WRL's brief, but anecdotal reports from CPB staff indicate that there have been fewer than average storm events during the monitoring period.

3. MECHANICAL SAND SHIFTING RECORDS

The salient was pre-built at the time of construction of the breakwater, assumed to be April 2004 for the purposes of calculation. Sand is transported from the salient to the south (backpassing) at the Semaphore Park erosion zone. Sand is also placed in the eroding area to the north of the breakwater (replenishment) from a sand source further to the north.

The following records of sand carting and placement were supplied by CPB:

- November 2005: 40,000 m^3 FROM salient to Semaphore Park erosion area to the south
- April 2006: 12,220 m^3 TO salient (downdrift area)
- November 2006: 10,000 m^3 TO salient (downdrift area).
- June 2007: 30,000 m^3 FROM salient to Semaphore Park erosion area to the south.

For the time period April 2004 to June 2007, the above rates equate to approximately:

- 22,000 m^3/year of backpassing from the salient to Semaphore park erosion zone to the south.
- 7,000 m^3/year of replenishment (from a sand source further to the north) of the downdrift area.

4. MEASURED BEACH CHANGE AND COMPARISONS WITH PREDICTIONS

4.1 Modelled Scenarios

Numerous shoreline change scenarios were modelled in Carley *et al.* (2001), however, none of the modelled scenarios exactly matched the eventuality of the trial breakwater history. As described in Section 3, the average extraction rates from the salient for backpassing to date have been 22,000 m^3/year which is similar to the rate of 28,000 m^3/year used in many model scenarios. As discussed in Section 2, most detailed modelling was undertaken for a breakwater crest of +1 m AHD (the original as-built level), whereas the present crest is -0.9 m AHD.

The average actual rate of downdrift replenishment has been 7,000 m^3/year. One modelling scenario was undertaken for downdrift replenishment rates of 0, 50,000, 70,000 and 90,000 m^3/year (Figure 8.31 Appendix E). Another scenario was undertaken within GENESIS modelling involving natural bypassing (south to north) of the salient of 33,000 m^3/year and for downdrift replenishment rates of 0, 10,000, 20,000, 30,000, 40,000 and 50,000 m^3/year (Figure 8.36 Appendix E), however, in this scenario, the salient did not fully form in the model due to the early imposition of natural bypassing.

In comparing available data with model predictions, the closest model scenarios to actuality have been used.

4.2 Available Data

Survey data was provided for the dates shown in Table 4.1. A plot of the survey tracks is shown in Appendix A.

Table 4.1
Available Surveys

Chronology of project	Detailed area survey	Profile survey	Satellite photo
			31/02/2002 00:54:21 UTC
	April 2002		
September 2003 mobilisation			
December 2003 sand carting halted			
February 2004 sand carting resumed			
		03/03/2004	
			25/10/2004 00:56:34 UTC
			18/12/2004 00:59:54 UTC
Early 2005 one tube left to place			
	February 2005		23/02/2005 00:58:45 UTC
		14 & 15/03/2005	
			06/05/2005 00:03:07 UTC
			22/08/2005 01:08:56 UTC
	November 2005		
November 2005 extraction of 40,000 m^3 of sand from salient			
	March 2006	02/03/2006	
			26/04/2006 01:14:47 UTC
			30/07/2006 01:23:16 UTC
			30/09/2006 01:15:26 UTC
	November 2006		28/11/2006 01:21:34 UTC
			29/01/2007 01:13:26 UTC
	February 2007		

4.3 Methodology for Analysis

4.3.1 Satellite Photos

The satellite photos show clear evidence of a salient formation and are shown in Appendix D. The tidal level at the time of the satellite photos was determined and found to vary widely. As the profile slope varied substantially in the lee of the breakwater, it was not readily practicable to transform the waterline in the satellite photo to a common contour. Given that good survey data was available, the survey data was used for quantitative analysis, with the satellite photos used as a qualitative guide.

4.3.2 Surveys

The surveys were imported into the software SURFER for analysis. Isopachs of change between surveys are shown in Appendix B and C. Following an initial analysis, the areas were divided up as shown in Figure 4.1. An accretion area was defined landward of the breakwater and south of 6,140,888 m North, which was the approximate northern transition point between beach widening and narrowing. An erosion area was defined landward of the breakwater and north of the erosion area to 6,141,640 m North, the approximate northward limit of beach narrowing. The shaded areas to the west and north shown in Figure 4.1 were excluded from the volume calculations (but included in the isopach analyses shown in Appendices B and C). This was because the shaded areas occupy a large area seaward of the structure – so that a small level change (including survey error or ripples) multiplied over the large area may mask or excessively reinforce the nearshore volume changes attributable to the breakwater. The landward limit of the analysed areas was determined by the extent of the surveys which may not have covered the full extent of dune change.

In addition to volume changes, the movement of the 0 m AHD contour was also tracked.

4.4 Beach Volume Changes

Beach volume changes for the areas shown in Figure 4.1 are shown in Figure 4.2. The discussion below focuses on the accretion area so that sustainable sand extraction quantities could be defined. The focus for analysis of the erosion area is on recession in Section 4.5.

For the accretion area between February 2005 and November 2005 (Figure 4.2) the equivalent annual accretion rate was approximately 45,000 m^3/year. Following the November 2005 survey, 40,000 m^3 of sand was mechanically removed from the salient (Section 3). The March 2006 survey (4 months later) can account for a removal of 22,000 m^3. The unaccounted for sand volume (18,000 m^3) is likely to be due to continued accretion in the salient between removal of sand (November 2005) and the March 2006 survey. Continued accretion at the previously observed rates (45,000 m^3/year) would account for 15,000 m^3 of the 18,000 m^3 volume previously unaccounted for.

Following the March 2006 survey, accretion rates have been smaller, at an equivalent annual rate of approximately 9,000 m^3/year. As described in Section 2, the as-built survey (21/12/2004) shows a crest level of the breakwater of approximately +1 m AHD, whereas the survey of November 2006 shows an average crest level of -0.9 m AHD. The timing of the lowering of the crest between these dates is not known, but based on GENESIS modelling undertaken in Carley *et al*. (2001) the observed lower accretion rate (9,000 m^3/year) is consistent with the low crest (-0.9 m AHD), while the observed higher accretion rate (45,000 m^3/year) is consistent with the original design crest (+1 m AHD). That is, an assumed date for crest lowering for the purposes of analysis is March 2006, though it is acknowledged that this lowering may not have been instantaneous.

4.5 Salient Dimensions, and Updrift and Downdrift Extent

The 0 m AHD contour determined from available surveys is shown in Figure 4.3. A close up of the salient is shown in Figure 4.4, together with the predicted and observed evolution of the salient width. As the salient was pre-built, an assumed start date of April 2004 has been used in Figure 4.4, whereas the actual pre-construction survey was in March 2002. Also shown in Figure 4.4 is the predicted salient width (without harvesting for backpassing), which shows a predicted salient width of approximately 100 m after 3 years (not pre-built), which asymptotes to 125 m by 10 years. This is slightly less than the 130 m to 140 m observed for the pre-built salient.

A summary of surveyed and modelled salient dimensions is shown in Table 4.2. The shoreline position relative to the March 2002 (latest pre-breakwater construction survey) is shown in Figure 4.5. It can be seen that the approximate updrift extent of accretion effects is 6,140,200 m N, which is approximately 500 m south of the centre of the breakwater. For the closest scenarios modelled in GENESIS (Appendix E, Figures 8.24, 8.29, 8.31 from Carley *et al.* 2001), updrift accretion for a distance of 475 m (at t = 3 years) was estimated for no backpassing and 500 m (at t = 4 years) for backpassing of 28,000 m^3/year.

The approximate downdrift extent of erosion effects (Figure 4.5) is 6,141,700 m N, which is approximately 1000 m north of the centre of the breakwater. For the closest scenarios modelled in GENESIS (Appendix E, Figures 8.24, 8.29, 8.31 from Carley *et al*. 2001), a distance of 675 m (at t = 3 years) was estimated for no backpassing or downdrift replenishment and 800 m (at t = 4 years) for backpassing of 28,000 m^3/year but no downdrift replenishment.

The survey data indicates a maximum downdrift recession of 60 m (with downdrift replenishment of 7,000 m^3/year). GENESIS modelling predicted 70 to 83 m for the closest modelled scenarios, albeit with no downdrift replenishment.

Table 4.2
Surveyed and Modelled Salient Dimensions

Variable	Dimension from Analysis of Survey Data	Closest Modelled Scenario GENESIS Figure from Carley *et al.*, (2001) and reproduced in Appendix E	GENESIS Model Prediction
Salient width (m)	130 m to 142 m (pre-built)	8.24 (not pre-built, no backpassing)	100 m at t = 3 years 125 m at t = 10 years
Updrift extent (m)	500 m	8.24 (no backpassing) 8.29, 8.31 (backpassing at 28,000 m^3/year)	475 m at t = 3 years 525 m at t = 4 years 775 m at t = 6 years 500 m at t = 4 years
Downdrift extent (m)	1000 m	8.24 (no backpassing, no downdrift replenishment) 8.29, 8.31 (backpassing at 28,000 m^3/year, no downdrift replenishment)	675 m at t = 3 years 775 m at t = 4 years 875 m at t = 6 years 800 m at t = 4 years
Maximum downdrift recession (m)	60 m	8.24 (no backpassing, no downdrift replenishment) 8.29 (backpassing at 28,000 m^3/year, no downdrift replenishment) 8.36 (backpassing at 28,000 m^3/year, natural bypassing of 33,000 m^3/year, downdrift replenishment of 10,000 m^3/year) 8.36 (backpassing at 28,000 m^3/year, natural bypassing of 33,000 m^3/year, no downdrift replenishment	70 m at t = 3 years 75 m at t = 4 years 83 m at t = 4 years 40 m @ t = 4 years 52 m @ t = 4 years

4.6 Profile Data

The profile data for three transects is shown in Figure 4.6. The location of the transects is shown in Appendix E Figure 8.22 (from Carley *et al.* 2001). In Figure 4.7, the 0 m AHD contour has been translated to a common X value for all three locations so that the profile gradients can be compared. It can be seen that the profile at Bower Road (in the lee of the breakwater) is generally flatter than the other profiles. The "one line" principle of GENESIS assumes a single beach profile for the model domain. Technically this flatter observed profile is a violation of the "one line" principle of GENESIS. It does not, however, obviate the model results, but is another illustration of model approximation and simplification of real world processes. This was discussed in Section 3.4 and Figure 3.5 (reproduced in Appendix E of this report) of Carley *et al*. (2001).

5. DISCUSSION OF MEASURED VERSUS PREDICTED PERFORMANCE

A detailed comparison of modelled versus measured performance was presented and summarised in Table 4.2. This comparison is discussed below. Note that numerous shoreline change scenarios were modelled in Carley *et al*. (2001), however, none of the modelled scenarios exactly matched the eventuality of the trial breakwater history.

The salient width (seaward distance) was predicted to be 100 m at 3 years and 125 m at 10 years in GENESIS predictions. The salient was pre-built so a direct comparison is difficult, however, in surveys its width was 130 to 142 m.

The extent of updrift accretion was predicted to be 475 to 525 m for various modelled scenarios over 3 to 4 years. The surveyed extent was 500 m.

The extent of downdrift recession was estimated to range from 675 m to 800 m for various 3 to 4 year modelled scenarios. The surveyed extent was 1000 m. The maximum downdrift recession was estimated to range from 40 to 80 m for various modelled scenarios (albeit with downdrift replenishment in the range 0 to 10,000 m^3/year), versus a surveyed extent of 60 m with 7,000 m^3/year of downdrift replenishment. That is, the modelling predicted the landward extent of recession, but slightly underpredicted the northward limit of recession.

For the original as-built crest level of +1 m AHD, the modelling predictions were that a sand extraction rate from the salient of 28,000 m^3/year was sustainable and that a rate of 40,000 m^3/year was marginally sustainable, albeit with major instability in the salient. Analysis of the survey data found this to be in the correct range, in that the pre-built salient was found to accrete at approximately 45,000 m^3/year during the initial 9 months of monitoring. The caution is that there has only been this 9 months of monitoring of the structure where it is believed that the crest was at +1 m AHD. For the lower crest (-0.9 m AHD) the accretion rate in the salient has been 9,000 m^3/year, which is far lower than intended extraction rates.

Overall, the measured performance has shown good agreement with the modelling. Variations between measured and predicted performance can be explained by the variability in natural processes, and simplifications and inaccuracies in the modelled wave climate, sediment transport and shoreline response.

6. RECOMMENDATIONS REGARDING TRIAL STRUCTURE

6.1 Sustainable Sand Extraction Volumes

A comprehensive monitoring program has been undertaken for this project, however, as with most data collection exercises involving natural processes, considerable change has occurred between measurements.

Sustainable sand extraction volumes are in the range 28,000 m^3/year to 45,000 m^3/year for the breakwater at its current location with a crest at +1 m AHD. 28,000 m^3/year has proven to be sustainable with removal of sand after approximately 18 months. The salient growth rate (and sand accumulation) is most rapid in the earlier stages of salient formation (Figure 4.4) before asymptoting towards a stable form with bypassing. Therefore higher sustainable rates (more than 28,000 m^3/year) may be able to be extracted, particularly if removal is more frequent than 18 months. Should the option of more frequent removal be pursued, a longer structure (say 300 m versus the present 200 m) could increase accretion volumes if the frequent removal of sand prevents the formation of a tombolo. The main caution with increased extraction rates is that a small (or no) salient may be more vulnerable to storm erosion should such events occur. Additional monitoring of the future structure (trial or permanent) would be needed to determine sustainability above extraction rates of 28,000 m^3/year.

On the basis of the analysed data, sustainable sand extraction rates would reduce to 9,000 m^3/year for the breakwater at its current location with the current crest at -0.9 m AHD, and may be less than this due to the increased impacts of large storm wave events, which anecdotally have not occurred during the monitoring period.

6.2 Location of Structure

Locating a permanent structure further north may reduce the downdrift erosion effects compared with the current structure, as the downdrift area is located further into the historically accreting area. Some guidance on the local sediment transport gradient is shown in Appendix E Figure 8.2 of Carley *et al*. (2001).

As described previously, the GENESIS modelling accurately predicted the structure's performance for the Bower Road location. Additional modelling scenarios were undertaken for breakwaters off Jervois Street (~450 m further north, Appendix E Figure 8.37) and Hart Street (~800 m north of Bower Road, Appendix E Figure 8.38). Similar performance was predicted between the Bower Road location and Hart Street, however, Bower Road is closer to the Semaphore Park erosion zone, which at the time, was considered to have lower sand carting costs. Within a revised sand shifting regime (Section 8 and DEH, 2005), this reduced distance for sand carting may be less advantageous, therefore a breakwater at Hart Street (instead of Bower Road) is likely to result in similar accretion volumes but reduced downdrift effects.

The Hart Street site is further from the Semaphore park erosion zone and provides less beach widening in the areas needed to the south of Bower Road, but with a suitable sand slurry system (DEH, 2005) it would result in reduced downdrift erosion and shorter carting distances from the vicinity of Semaphore jetty.

7. DESIGN PARAMETERS FOR PERMANENT STRUCTURE

7.1 Water Levels

It is noted that as of 1 January 2001, the chart datum for Adelaide was raised by 0.271 m, from –1.723 m AHD to –1.452 m AHD. All levels in this report are in AHD unless noted otherwise. Water levels for Port Adelaide (outer harbour) are given in Tables 7.1 and 7.2. The astronomical tide levels were obtained from Australian National Tide Tables (2007). Extreme water levels were obtained from Wynne *et al*., (1984), and Riedel and MacFarlane (1999). Wynne *et al*. (1984) used an analysis of tide gauge records performed by the Department of Marine and Harbours for the period 1915 to 1967 and noted that the maximum recorded tidal anomaly was 1.6 m on 2 December 1973 at low tide. They also noted that larger tidal anomalies occurred at low tide due the greater propensity for wind setup.

Substantial discussion on global sea level rise is contained in IPCC (2001) and IPCC (2007). Current Coast Protection Board policy is to plan for an absolute sea level rise of 0.3 m for 2050 plus an allowance for subsidence depending on location (0.1 m for Port Adelaide). Strictly speaking, Australian Height Datum is an absolute vertical datum, so the estimated water levels for 2050 shown in Table 7.2 include only the 0.3 m rise for absolute sea level rise.

The design water levels are similar from the two analyses, with the values of Riedel and MacFarlane (1999) adopted for this study, as they are the joint probability water levels for the accompanying ARI wave height.

Table 7.1
Astronomical Tidal Water Levels for Adelaide

	Water Level (m relative to datum)		
Description	**Chart Datum (m CD)**	**Australian Height Datum (m AHD)**	**AHD estimate for 2050 with 0.3 m absolute rise**
LAT - lowest astronomical tide	0.0	**-1.45**	-1.15
MLWS - mean low water springs	0.3	**-1.14**	-0.84
MLWN - mean low water neaps	1.3	**-0.15**	0.15
MSL - mean sea level	1.3	**-0.15**	0.15
MHWN - mean high water neaps	1.3	**-0.15**	0.15
MHWS - mean high water springs	2.3	**1.15**	1.45
HAT - highest astronomical tide	2.8	**1.65**	1.95

Table 7.2
Extreme Design Water Levels for Adelaide

Extreme Event	**Water Level (m AHD)**			
Average Recurrence Interval (ARI, years)	Wynne *et al.*, (1984)	Riedel and MacFarlane (1999) Joint probability with same ARI design waves	**Present values adopted for this study**	2050 values adopted for this study with 0.3 m absolute rise*
1	1.92	1.50	**1.50**	1.80
5	2.14	1.95	**1.95**	2.25
10	2.25		**2.05**	2.35
20	2.33	2.15	**2.15**	2.45
50	2.47		**2.30**	2.60
100	2.51	2.35	**2.35**	2.65

Notes: * See discussion on absolute and relative sea level rise above

7.2 Wave Climate and Design Wave Conditions

7.2.1 Offshore Waves

Extreme design wave and water level conditions need to be used for stability calculations for the proposed structure. Design offshore waves are listed in Table 7.3. The values presented in Riedel and MacFarlane (1999) are slightly higher than the earlier studies and were derived for the Holdfast Shores development – these values have been adopted in this study. It should be noted that the significant wave height (Hs) is given in Table 7.3. Individual waves within the design storm will exceed Hs. However, for design of non-rigid/rubble structures the use of significant wave height or $H_{10\%}$ (≈ 1.27 Hs) is recommended (SPM, 1984), whereas for rigid structures the use of $H_{1\%}$ is recommended, which is approximately 1.67 Hs. As described below, depth limitations may prevent such waves reaching a structure.

Table 7.3
Extreme Offshore Waves for Adelaide

Extreme Event	**Hs (m)**		
ARI (years)	Wynne *et al.* (1984), Kinhill *et al.* (1983)	Riedel and MacFarlane (1999) Joint probability with same ARI water level	Values adopted for this study
1		2.1	2.1
5		2.4	2.4
10			2.7
20		2.9	2.9
50	3.0*		3.1
100		3.4	3.4

* Described as ".... return period greater than 50 years" and used as design wave.

7.2.2 Depth Limited Waves at the Structure

The local ratio of wave height (H) to water depth (d) is denoted as γ with the ratio at wave breaking denoted γ_b. Inside the surf zone the water depth includes wave setup, however, at high water levels, the offshore breakwater is near the seaward limit of the surf zone. SPM (1984) Figure 7-4 gives the following γ_b values for the range of water depths and wave periods applicable during extreme events at Semaphore Park:

- γ_b = 0.78 for horizontal bed
- γ_b = 0.85 for bed slope of 1V:100H (varies with water depth and wave period).

Nelson (1999) has presented persuasive arguments that for bed slopes of 1V:100H and flatter, the breaker index curves in SPM (1984) are excessively conservative. He contended that for horizontal beds γ_b = 0.55, and that the value recommended in SPM (1984) of 0.78 is based on incorrect application of solitary wave theory which has resulted in excessively conservative designs. Bijker (1971) has also contended that in storm conditions with strong onshore winds the breaker index approaches 0.5, and this was used as the basis of nearshore design waves in Kinhill Stearns *et al.* (1983).

Offshore and design nearshore wave heights adopted for this study are presented below in Table 7.4. These heights are for a breaker index γ_b of 0.85, bed slope of 1V:100H and assume a combined refraction and shoaling coefficient of 1.0 – the adopted design wave height for armour stability is the lesser of $H_{10\%}$ and Hb. The typical bed level just seaward of the breakwater is -2 m AHD, with an allowance of 0.5 m added for scour, giving a design bed level of -2.5 m AHD. The wave heights presented in Table 7.4 are to assist in predicting the stability of a permanent rock armoured breakwater. The 2050 values assume a sea level rise of 0.3 m.

Table 7.4
Extreme Nearshore Waves for Adelaide
(based on γ_b = 0.85 and Kr Ks = 1.0)

ARI (years)	Offshore Hs (m)	Offshore $H_{10\%}$ (m)	Present day SWL (m AHD)	Present day Hb (m)	2050 Hb (m)	Present day Design H	2050 Design H
1	2.1	2.7	1.50	3.4	3.7	**2.7**	**2.7**
5	2.4	3.1	1.95	3.8	4.0	**3.1**	**3.1**
10	2.7	3.4	2.05	3.9	4.1	**3.4**	**3.4**
20	2.9	3.7	2.15	4.0	4.2	**3.7**	**3.7**
50	3.1	3.9	2.30	4.1	4.3	**3.9**	**3.9**
100	3.4	4.3	2.35	4.1	4.4	**4.1**	**4.3**

7.3 Armour Size

An overview of rock armour structures having similar exposure the Semaphore Park breakwater has been provided by CPB staff. These structures have been in place on the Adelaide coast for up to 20 years or more and are listed in Table 7.5.

Table 7.5
Armour Size of Comparable Adelaide Structures

Location	Age	Primary Armour Size	Comment
North Haven Marina Breakwater	>20 years	5 tonnes	Stable
Holdfast Shores (Glenelg) Offshore Breakwater	>10 years	Primary 6 tonnes Secondary 1.5 to 5 tonnes	Some movement noted, but believed to be due to construction techniques
Holdfast Shores Breakwater (perpendicular to shore)	>10 years	Trunk 6 tonnes Head 8 tonnes	Stable

Numerous methods exist for calculating armour size and stability, many of which are presented in CEM (2003). It is noted that for a crest level of +1 m AHD the structure is substantially submerged for the extreme high water levels presented above, though not sufficiently submerged for the application of most toe armour sizing techniques. The following armour size estimates assume an armour density of 2700 kg/m^3.

SPM (1984) techniques using the Hudson Equation, estimates a required primary armour of approximately 15 tonnes, whereas the SPM (1977) techniques using the Hudson Equation estimate primary armour of approximately 4 tonnes for the trunk. Application of the method of van der Meer (Delft, 1990) for emergent structures estimates a primary armour size of approximately 5 tonnes for the trunk. Proper consideration of the submergence of the Semaphore Park breakwater using the method of van der Meer (1991) indicates that primary armour of 2 tonnes would be stable.

It can be seen that the various desktop techniques result in widely differing predicted armour sizes. Most authoritative references recommend physical model testing be undertaken in the detailed design stage. In light of the performance of existing Adelaide structures, provided that the a minimum armour size of 5 tonnes is used, model testing may not be necessary, however, it may assist with properly understanding the stability of the crest and ends (heads) of the breakwater.

7.4 Dimensions, Quantities and Costs of Permanent Breakwater

The input variables and dimensions of a single permanent breakwater are shown in Table 7.6. The recommended dimensions (Table 7.6) are a crest length of 200 m, a crest elevation of +1 m AHD and a minimum crest width of three armour stones.

The thickness of the double layer of primary armour shown in Table 7.6 means that even if a permanent breakwater was to be constructed in the same place, the (remains of the) geocontainer trial breakwater would need to be removed, as there is insufficient height available. Therefore, the precise location of the permanent breakwater is not physically dependent on the trial structure.

The quantities and indicative costs for a permanent breakwater of the recommended dimensions are shown in Table 7.7. It can be seen that an indicative cost estimate for a permanent breakwater with 5 tonne rock is $1.4 million.

Table 7.6
Summary of Permanent Breakwater Design Parameters

Variable	Symbol	Value
Crest length		200 m
Crest level		+1 m AHD
Toe level		-3 m AHD
Typical bed level		-2 m AHD
Scoured bed level		-2.5 m AHD
Rock density	ρ_{rock}	2700 kg/m^3
Porosity		0.37
Primary armour mass	M50	5 tonnes
Primary armour cube side equivalent	Dn50	1.3 m
Thickness of two layers (primary armour)		2.5 m
Minimum crest width		3.9 m
Side slope		1V:1.5H
Secondary armour	M50	500 kg
Secondary armour cube side equivalent	Dn50	0.6
Thickness of two layers (secondary)		1.1

Table 7.7
Permanent Breakwater Quantities and Costs

Quantities	Unit	Quantity	Rate ($ ex GST)	Amount ($ ex GST)
Design and documentation	Item	1	20,000	20,000
Contractor mobilisation/demobilisation	Item	1	50,000	50,000
Bed preparation	m^2	3,180	4	12,720
Geotextile underlay (supply and place)	m^2	3,180	15	47,700
Secondary armour - supply	tonnes	3,650	30	109,500
Secondary armour - place	tonnes	3,650	15	54,750
Primary armour - supply	tonnes	15,300	40	612,000
Primary armour - place	tonnes	15,300	20	306,000
Total Cost				1,212,670

8. COST COMPARISON OF ALTERNATIVES

The following three alternatives are compared below:

1. A field of five permanent breakwaters similar in size to the “as designed” Bower Road structure, with approximately 200 m gaps between each breakwater.
2. A single permanent breakwater located off Bower Road to act as a sand trap, together with sand pumping from the salient into the Semaphore Park erosion zone.
3. A single permanent breakwater located off Hart Street to act as a sand trap, together with sand pumping from the salient into the Semaphore Park erosion zone.

For all three options, replenishment of the downdrift area of the salient would still be undertaken by sand carting from near Semaphore Jetty, at a rate of 10,000 m^3/year for alternatives 1 and 2, and a rate of 5,000 m^3/year for alternative 3. Options 2 and 3 involve pumping sand (40,000 m^3/year) from the salient to the Semaphore Park erosion zone.

The sand pumping systems are based on those investigated in Department for Environment and Heritage (2005) and would extend from the existing breakwater to the Semaphore Park erosion zone, a distance of 1 km to 1.6 km (1.5 km used for costing) for a breakwater at Bower Road, and 1.8 to 2.4 km (2.3 km used for costing) for a breakwater at Hart Street. The cost for these systems would still involve one permanent breakwater. The costs for permanent breakwaters are detailed in Section 7. It has been assumed that the field of five breakwaters would be constructed in years 2 and 3 of the analysis.

The component costs are shown in Table 8.1, with the total costs using a net present value analysis over 20 years shown in Table 8.2. Interest rates of 4%, 7% and 10% have been used.

It can be seen that both the single breakwater/sand pumping schemes involve a lower net present value cost. The five breakwaters option represents a higher capital cost but reduced ongoing maintenance. Once implemented it would be less dependent on the commitment of ongoing funds, correct scheduling and mechanical equipment. The option of a single breakwater at Hart Street offers a slightly lower net present cost than a single breakwater at Bower Road, as the reduced sand carting costs (from Semaphore Jetty to salient) outweigh the slightly longer pipeline costs to the Semaphore Park erosion zone. This cost advantage for the Hart Street breakwater is dependent on not needing a sand pumping booster station.

Table 8.1
Component Costs of Alternatives

Option and Timing of Capital Works	Capital Cost ($ ex GST)	Annual Operating cost ($/year ex GST)
Option 1 – Field of Five Breakwaters		
Field of five breakwaters – two in year 2 and three in year 3	$6,000,000	
Sand carting from Semaphore Jetty to downdrift erosion area of salient – Bower Road (10,000 m^3/year over 1.4 km @ $5/$m^3$/km)		$70,000
Option 2 – Single Breakwater at Bower Road		
Construction of single 200 m long rock breakwater – year 2	$1,200,000	
Pipeline costs for sand pumping (1.5 km @ $400/m) – year 2	$600,000	
Sand pumping station ("Sand shifter") – year 2	$950,000	
Sand pumping costs (for 40,000 m^3/year over 1.5 km)		$17,000
Sand carting from Semaphore Jetty to downdrift erosion area of salient – Bower Road (10,000 m^3/year over 1.4 km @ $5/$m^3$/km))		$70,000
Option 3 – Single Breakwater at Hart Street		
Construction of single 200 m long rock breakwater – year 2	$1,200,000	
Pipeline costs for sand pumping (2.3 km @ $400/m) – assumed no booster station (nominal requirement at 2.2 km intervals) – year 2	$920,000	
Sand pumping station ("Sand shifter") – year 2	$950,000	
Booster station – assumed nil (nominal requirement at 2.2 km intervals)	-	
Sand pumping costs (for 40,000 m^3/year over 2.3 km) – assumed no booster station		$17,000
Sand carting from Semaphore Jetty to downdrift erosion area of salient – Hart Street (5,000 m^3/year over 0.6 km @ $5/$m^3$/km))		$15,000

Table 8.2
Net Present Value Analyses of Alternatives over 20 years

Option	Net present cost ($ Millions) for discount rate		
	4%	**7%**	**10%**
Field of five breakwaters	6.4	5.8	5.3
Single permanent breakwater at Bower Road plus sand pumping ("Sand shifter")	2.7	2.3	2.1
Single permanent breakwater at Hart Street plus sand pumping ("Sand shifter")	2.4	2.2	2.0

9. CONCLUSIONS

9.1 Reason for the Study

Adelaide's beaches have had a long history of erosion. This led to a trial breakwater being constructed from geotextile tubes at Semaphore Park to trap sand by forming a salient. The sand can be extracted and used for replenishment of eroding beaches to the south. For this project, sand carting commenced in September 2003 and the breakwater construction commenced in April 2004. The offshore breakwater has been monitored by regular surveys since construction. This report details analysis of the monitoring data and additional information to determine the performance of the trial structure.

9.2 Measured versus Predicted Performance

The as-built crest level was +1 m AHD, which later reduced to -0.9 m AHD. When the crest level was inferred to be at the original level, the salient size and shape, and extent of updrift and downdrift effects were well approximated by the original modelling. Sustainable sand yields were found to be in the range 22,000 to 40,000 m^3/year.

The performance with the crest at -0.9 m AHD was also predicted in the original modelling, however, this reduced sustainable sand yields to a maximum of 9,000 m^3/year.

9.3 Recommendations for Permanent Structure and Sustainable Sand Yields

The trial breakwater has largely performed as intended, so based on a technical assessment a permanent structure is warranted. The crest should be at the original intended design level of +1 m AHD. Based on preliminary design, the rock size needed to make the breakwater permanent would exceed the available space above the geotextile tube structure, and therefore the trial structure needs to be removed. This means that the permanent structure need not be located in the original Bower Road location.

If the permanent structure is located off Hart Street, this would reduce the downdrift erosion and the required sand carting distance and quantities from 10,000 m^3/year to 5,000 m^3/year, subject to ongoing monitoring. This would result in greater distance between the breakwater and the Semaphore Park erosion area to the south, which would result in higher costs if the sand was to be carted in trucks. However, for a sand pumping system, this distance is less than that needed for a booster station, so the additional pipe costs (between the salient and the Semaphore Park erosion zone) are more than offset by the reduced distance and sand carting quantities between Semaphore Jetty and downdrift of the salient (see below). Sustainable sand yields for harvesting from the salient and pumping the sand to the Semaphore Park erosion area to the south are estimated to be 40,000 m^3/year subject to ongoing monitoring, provided that the salient is harvested regularly so that it is maintained in a small state. Accretion volumes in the salient reduce as its size increases.

9.4 Costs

A permanent breakwater designed using desktop techniques and comparison to other stable coastal structures around Adelaide is estimated to cost $1.2 Million subject to detailed design, which may require physical model testing.

The following three alternatives are compared below:

1. A field of five permanent breakwaters similar in size to the "as designed" Bower Road structure, with approximately 200 m gaps between each breakwater.
2. A single permanent breakwater located off Bower Road to act as a sand trap, together with sand pumping from the salient into the Semaphore Park erosion zone.
3. A single permanent breakwater located off Hart Street to act as a sand trap, together with sand pumping from the salient into the Semaphore Park erosion zone.

For all three options, replenishment of the downdrift area of the salient would still be undertaken by sand carting from near Semaphore Jetty, at a rate of 10,000 m^3/year for alternatives 1 and 2, and a rate of 5,000 m^3/year for alternative 3. Options 2 and 3 involve pumping sand (40,000 m^3/year) from the salient to the Semaphore Park erosion zone.

The net present costs at 7% discount rate over 20 years are:

1. Field of five breakwaters: $5.8 Million.
2. Single permanent breakwater at Bower Road: $2.3 Million.
3. Single permanent breakwater at Hart Street: $2.2 Million.

10. REFERENCES/BIBLIOGRAPHY

Bijker E.W., (1968), "Littoral Drift as a Function of Waves and Current", *Proceedings 11th Conference on Coastal Engineering,* Vol. I, London, England, pp 451-435.

Bijker E.W., (1971), "Longshore Transport Computations", *Journal of the Waterways, Harbors and Coastal Engineering Division,* Proceedings of the American Society of Civil Engineers, November 1971.

Carley, J.T., Cox, R.J. and Jackson, L.A., (2001), "Semaphore Park Offshore Breakwater Optimisation and Detailed Design", *WRL Technical Report No 01/24,* Water Research Laboratory, University of New South Wales.

CERC (1984), *Shore Protection Manual, Volume I and Volume II.* US Army Corps of Engineers, Coastal Engineering Research Center (CERC), Vicksburg, USA.

CES, (2000), *Semaphore Park Protection Strategy Review 1999-2000*. Coastal Engineering Solutions (CES) for Coast and Marine Section, Environment Protection Agency, South Australia.

Culver, R. (1970), *Final Summary Report on Beach Erosion Studies.* University of Adelaide, Department of Civil Engineering. Commissioned by the Seaside Councils Committee and the State Government of South Australia.

Delft (1990), "Breakwat Design Tool for Rubble Mound Structures", Delft Hydraulics, The Netherlands.

Department for Environment and Heritage (2005), Adelaide's Living Beaches A Strategy for 2005-2025, *Technical Report,* Government of South Australia.

EPA, (1999), *Semaphore Park Foreshore Protection Strategy Re-examination: Background and Alternatives*. Environment Protection Agency, Government of South Australia.

IPCC, (1996). *Climate Change 1995- The Science of Climate Change,* The United Nations Intergovernmental Panel on Climate Change Working Group I, Cambridge University Press, UK.

IPCC, (2001), *Technical Summary of the Working Group I Report and Summary for Policymakers,* The United Nations Intergovernmental Panel on Climate Change, Cambridge University Press, UK.

Kinhill Stearns Engineers & Riedel & Byrnes Consulting Engineers. (1983), *Adelaide Coast Protection Alternatives Study*. Kinhill Stearns Engineers and Riedel & Byrne Engineers Pty Ltd.

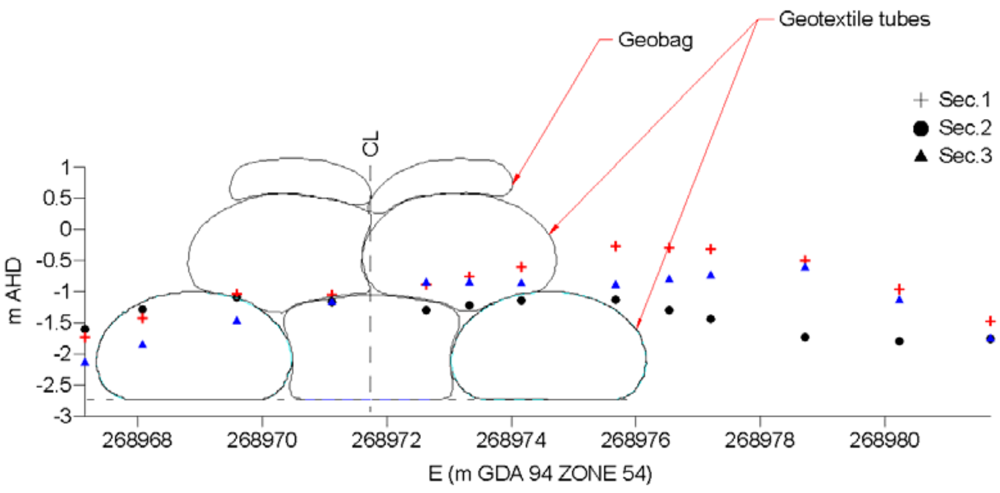
Nelson R.C., (1999), "A Comparison of Two Design Wave Criteria", *Coasts and Ports '99- Challenges and Directions for the New Century- Proceedings of the 14th Australasian Coastal and Ocean Engineering Conference and the 7th Australasian Port Conference,* National Committee on Coastal and Ocean Engineering and Institution of Engineers Australia, Vol. 2, pp 456-461.

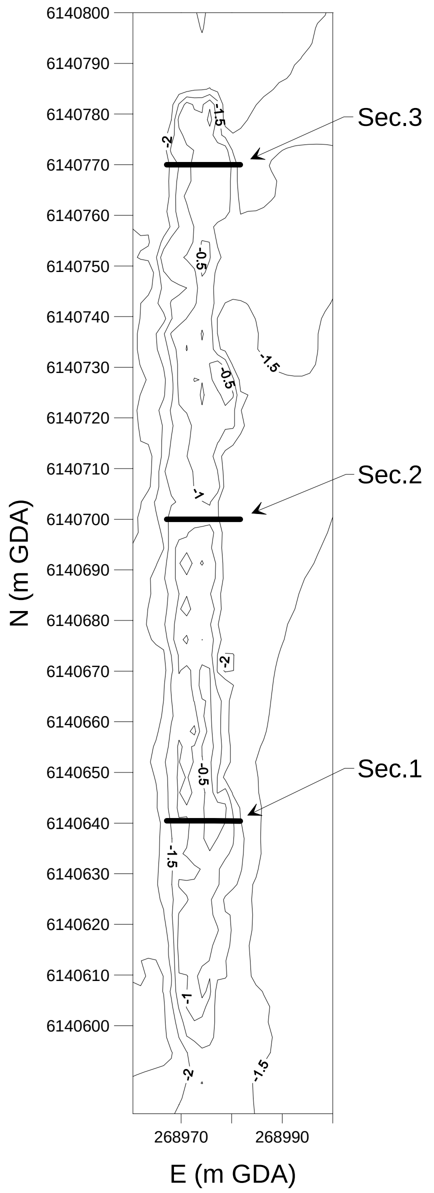
Riedel P. and MacFarlane B., (1999), "Holdfast Shores and West Beach, S.A.- Coastal Process Monitoring". *Coasts and Ports '99- Challenges and Directions for the New Century- Proceedings of the 14th Australasian Coastal and Ocean Engineering Conference and the 7th Australasian Port Conference,* National Committee on Coastal and Ocean Engineering and Institution of Engineers Australia, Vol. 2, pp 574-579.

Townsend M., (1999), *Semaphore Park Foreshore- Strategy Review Stage 1: Background and Options for Protection.* Coast and Marine Section, EPA, SA.

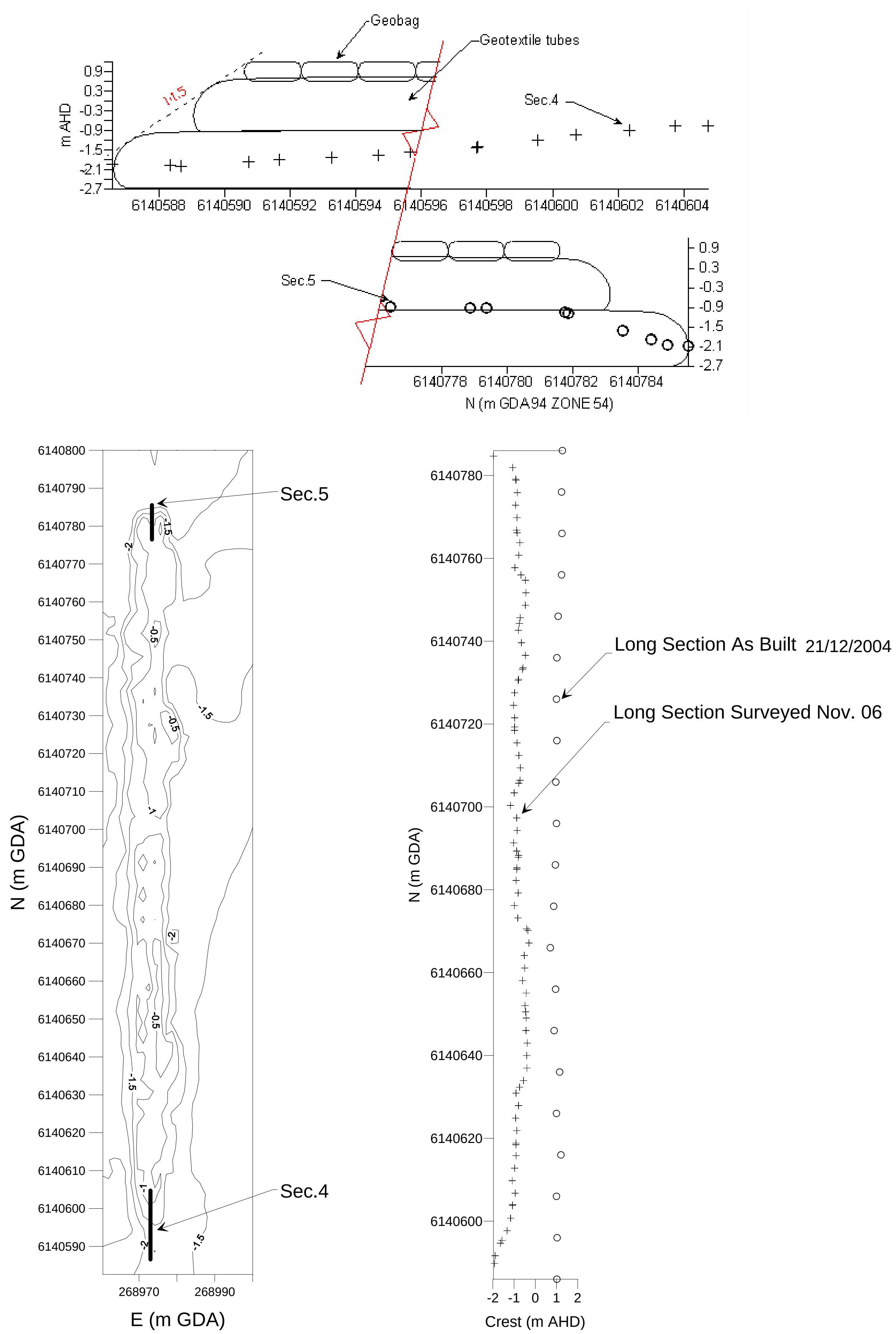
van der Meer, J. W. (1991), "Stability and Transmission at Low-Crested Structures," Delft Hydraulics Publication No. 453, Delft Hydraulics Laboratory, The Netherlands.

Wynne A.A, Fotheringham D.G., Freeman R., Moulds B., Penney S.W., Petrusevics P., Tucker R. & Ellis D.P., (1984), *Adelaide Coast Protection Strategy Review*. The Coastal Management Branch, Conservation Programmes Division, Department of Environment and Planning, South Australia. Prepared for The Coast Protection Board South Australia.

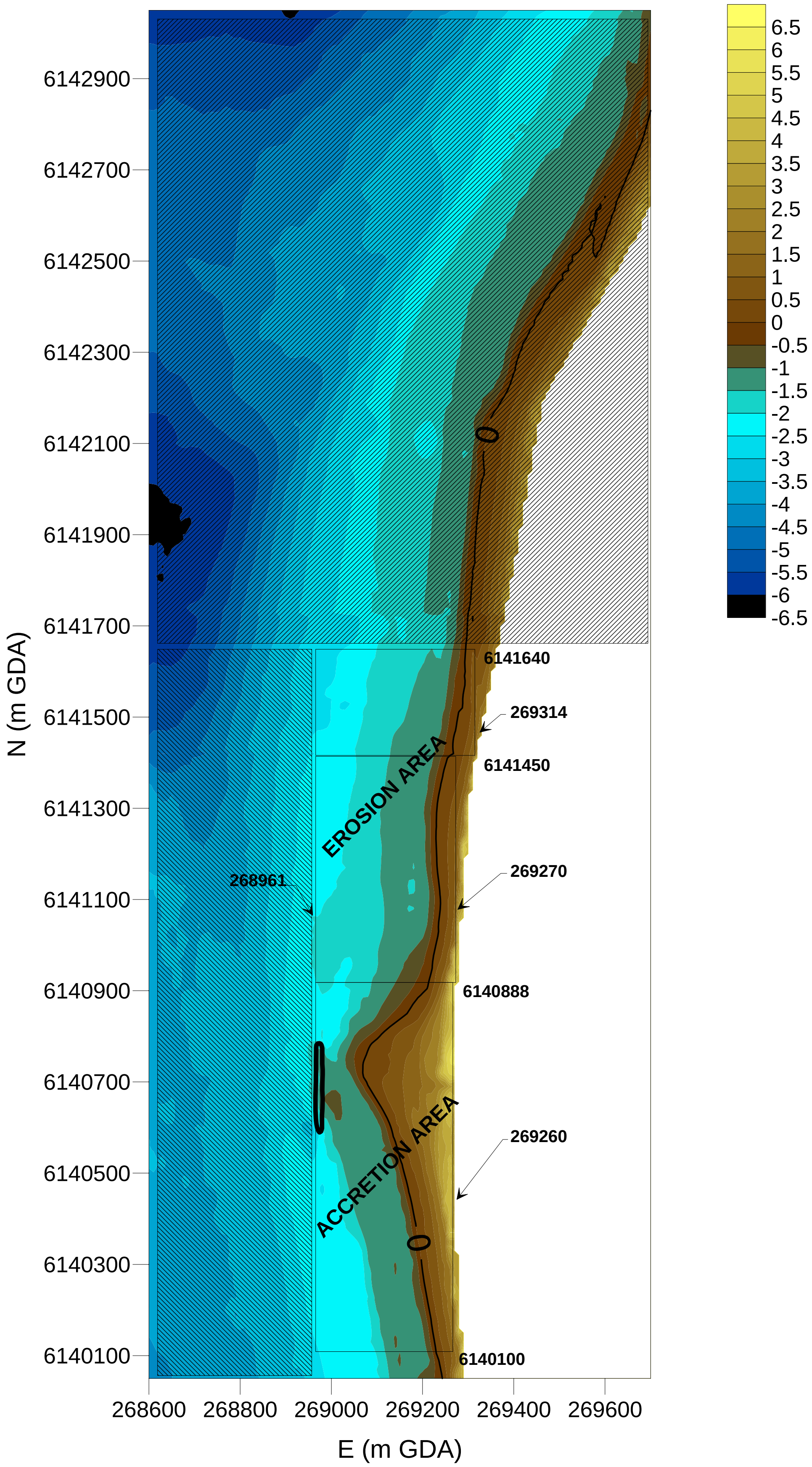




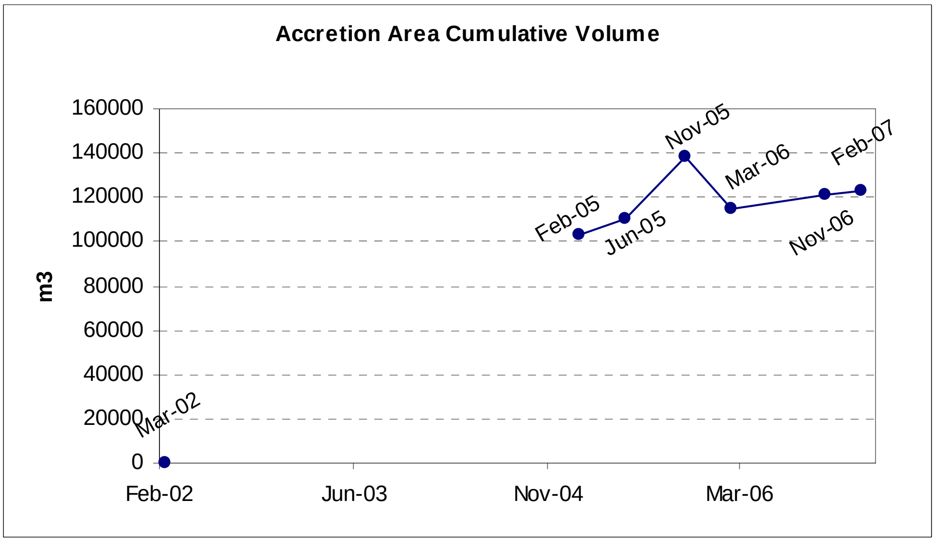
WRL Report No. 2007/14	DESIGNED & PRESENT BREAKWATER CROSS SECTIONS	Figure 2.1

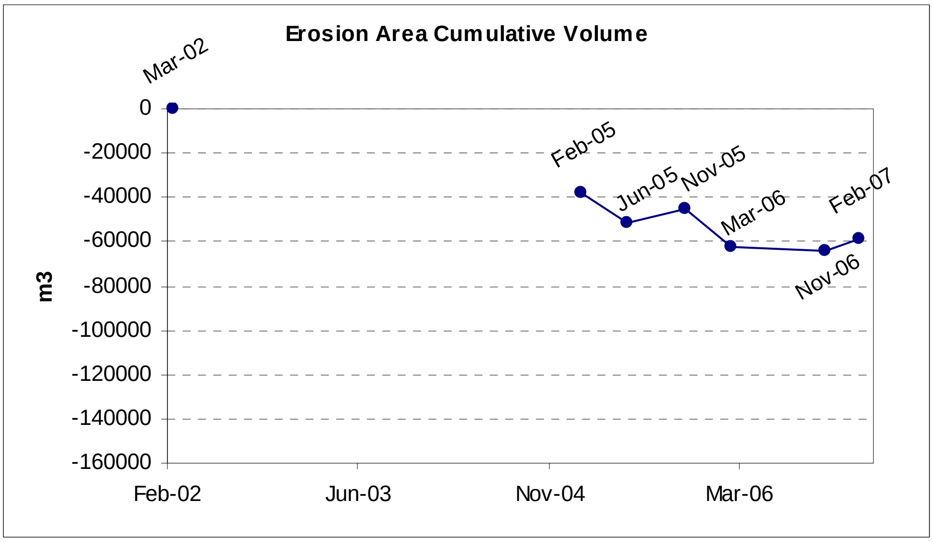


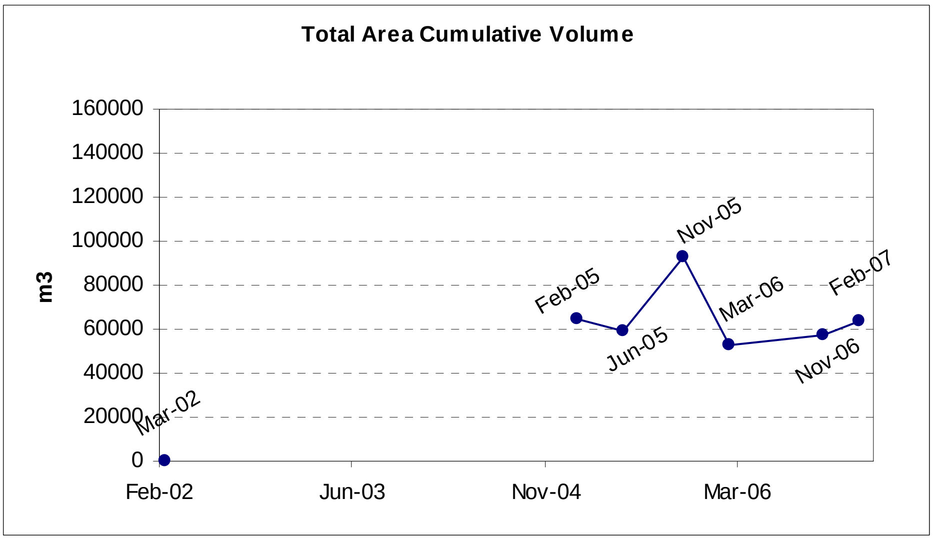
WRL Report No. 2007/14	DESIGNED, AS BUILT & PRESENT BREAKWATER LONG SECTIONS	Figure 2.2



WRL Report No. 2007/14	EROSION / ACCRETION AREAS	Figure 4.1

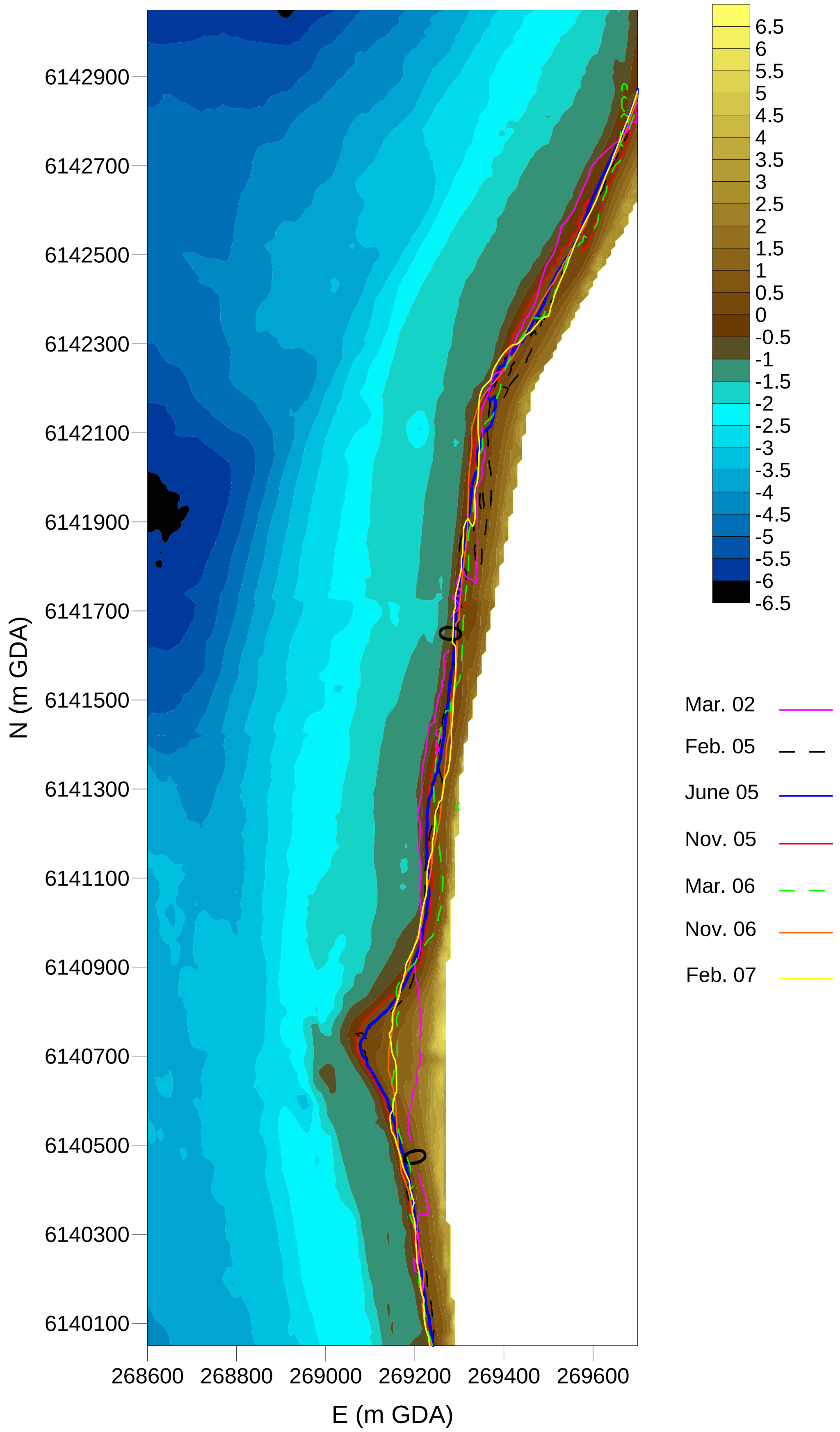




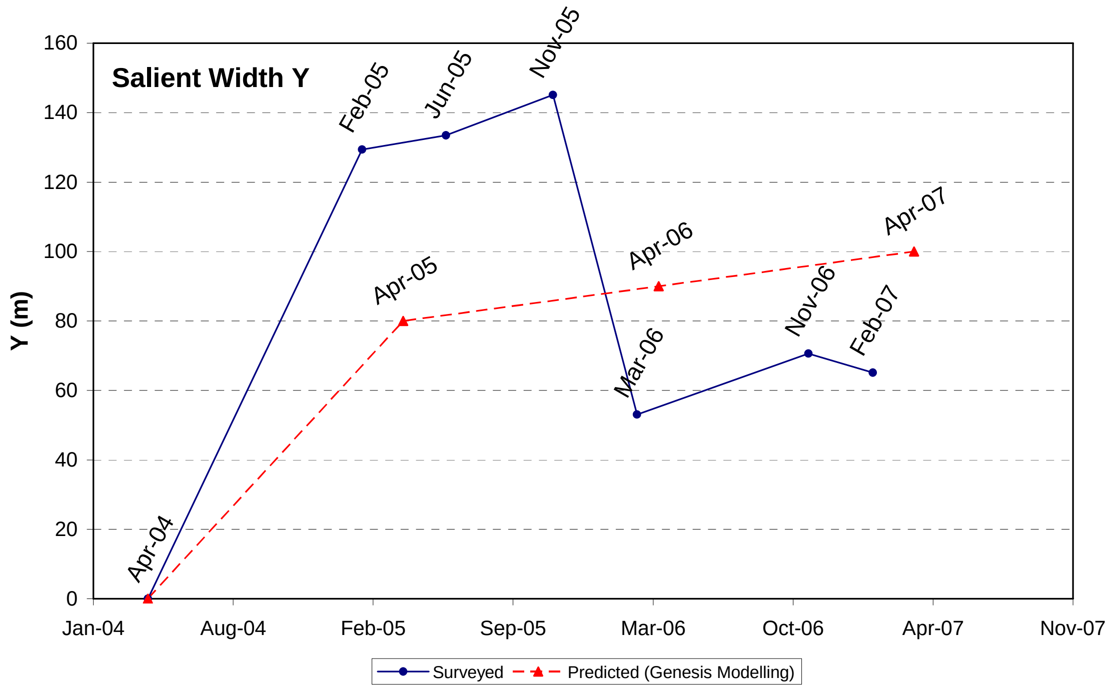


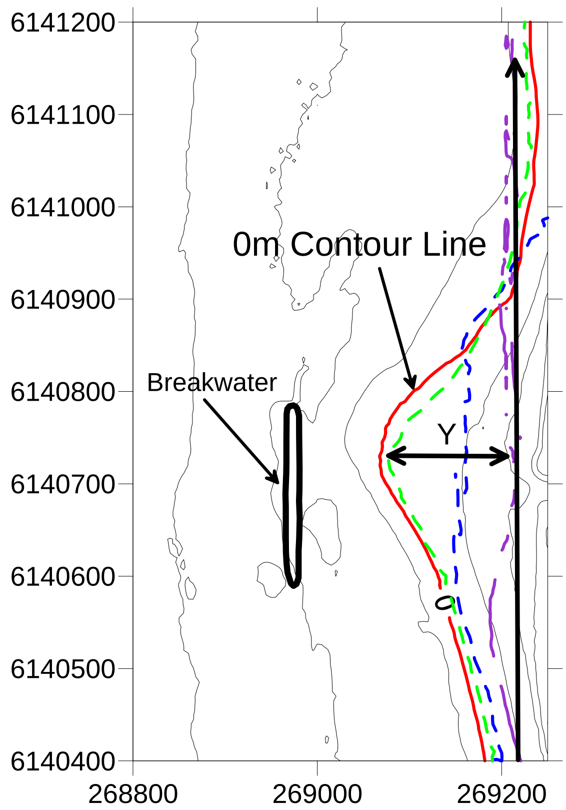
* March 2002 is the latest survey available before construction of the breakwater

WRL Report No. 2007/14	BEACH VOLUMES CHANGES	Figure 4.2

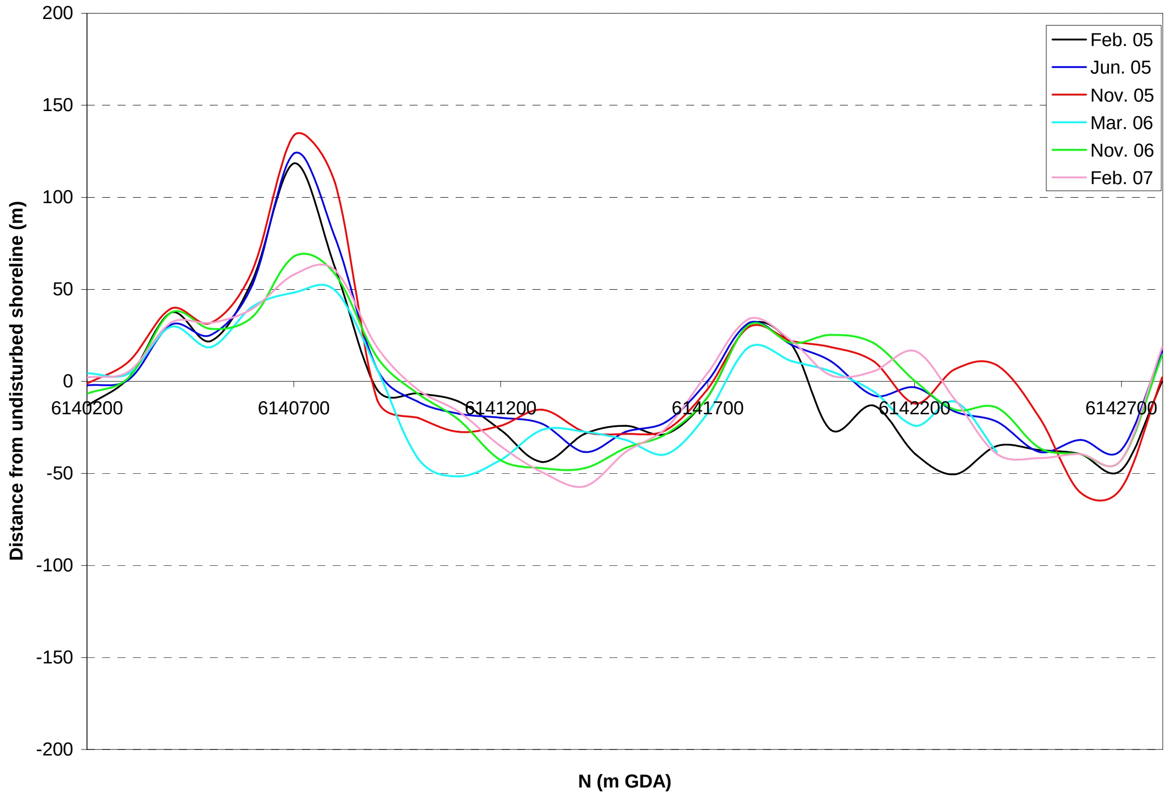


WRL Report No. 2007/14	SURVEYED SALIENT EVOLUTION	Figure 4.3

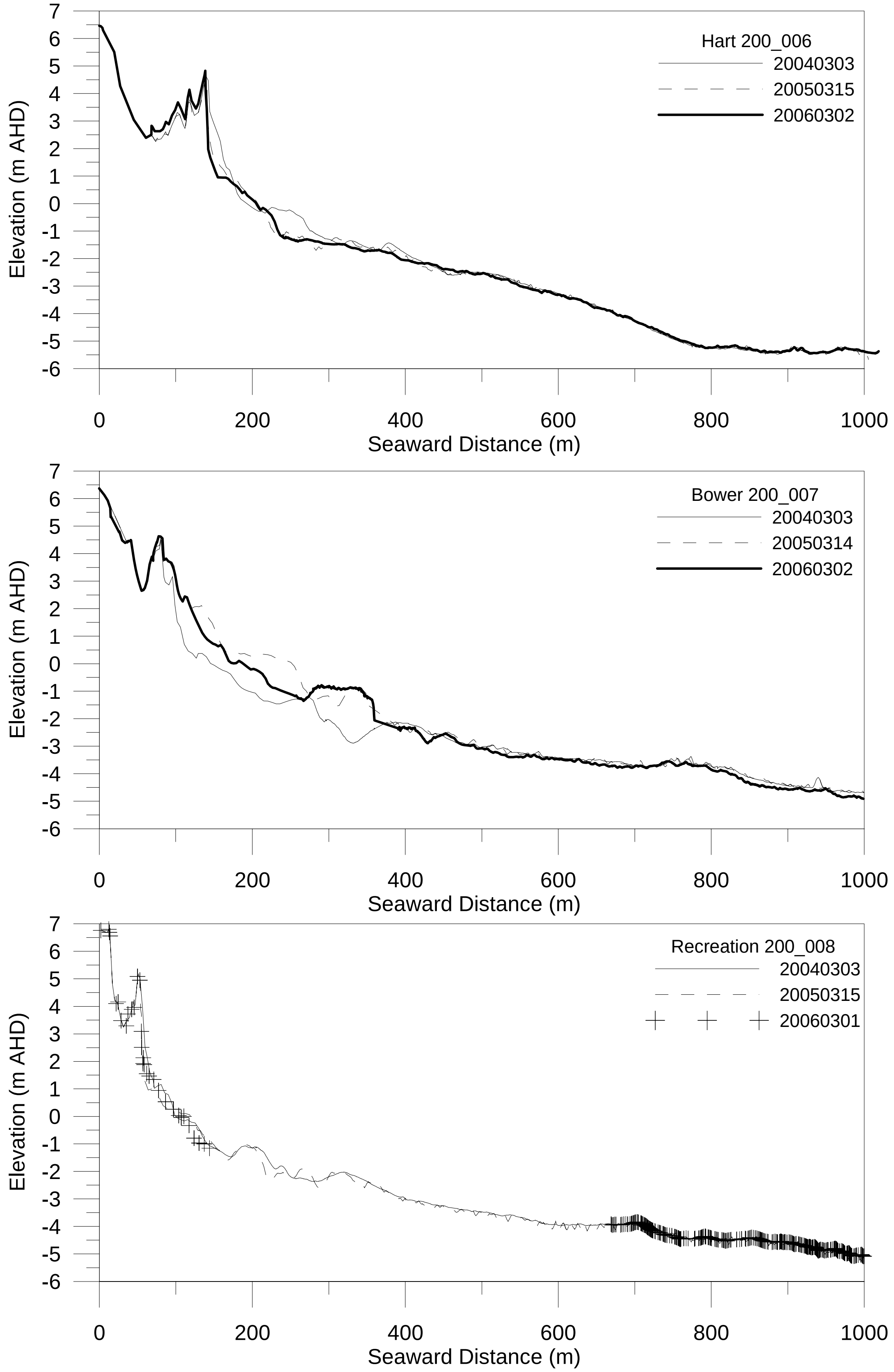




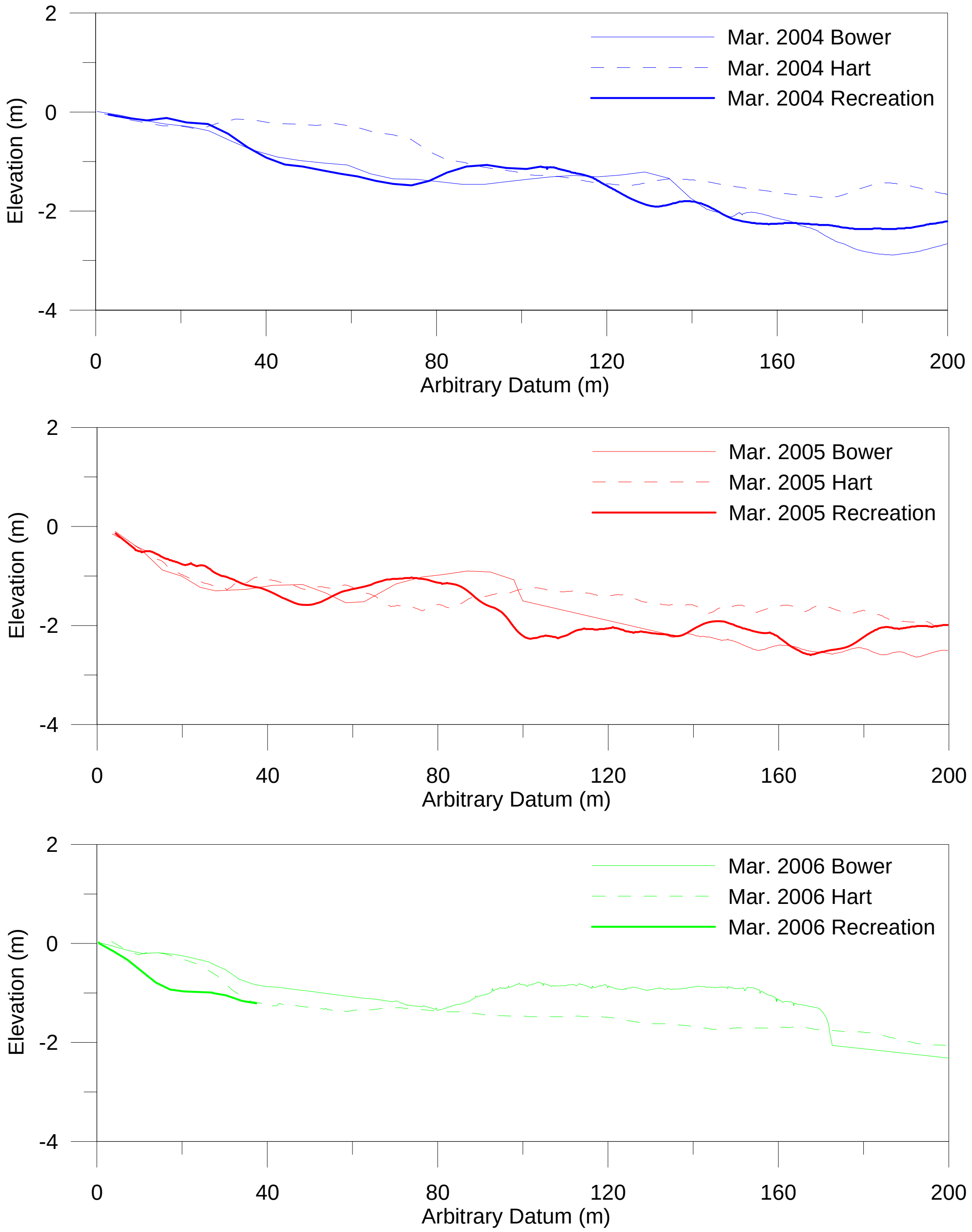
WRL Report No. 2007/14	SALIENT WIDTH ANALYSIS	Figure 4.4



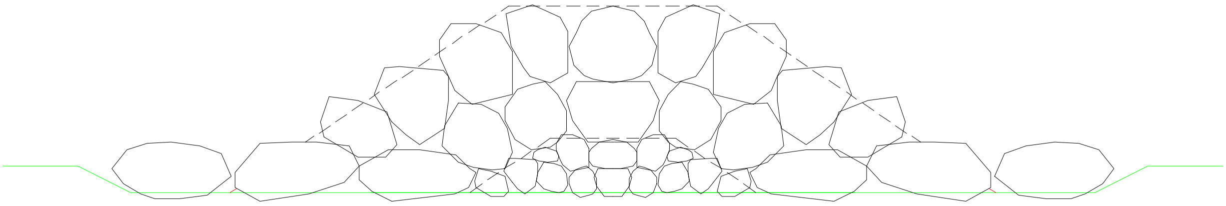
WRL Report No. 2007/14	SHORELINE CHANGES FROM MARCH 2002	Figure 4.5



WRL Report No. 2007/14	PROFILE DATA 2004 -2006	Figure 4.6



WRL Report No. 2007/14	NEARSHORE PROFILE SLOPES 2004 -2006	Figure 4.7

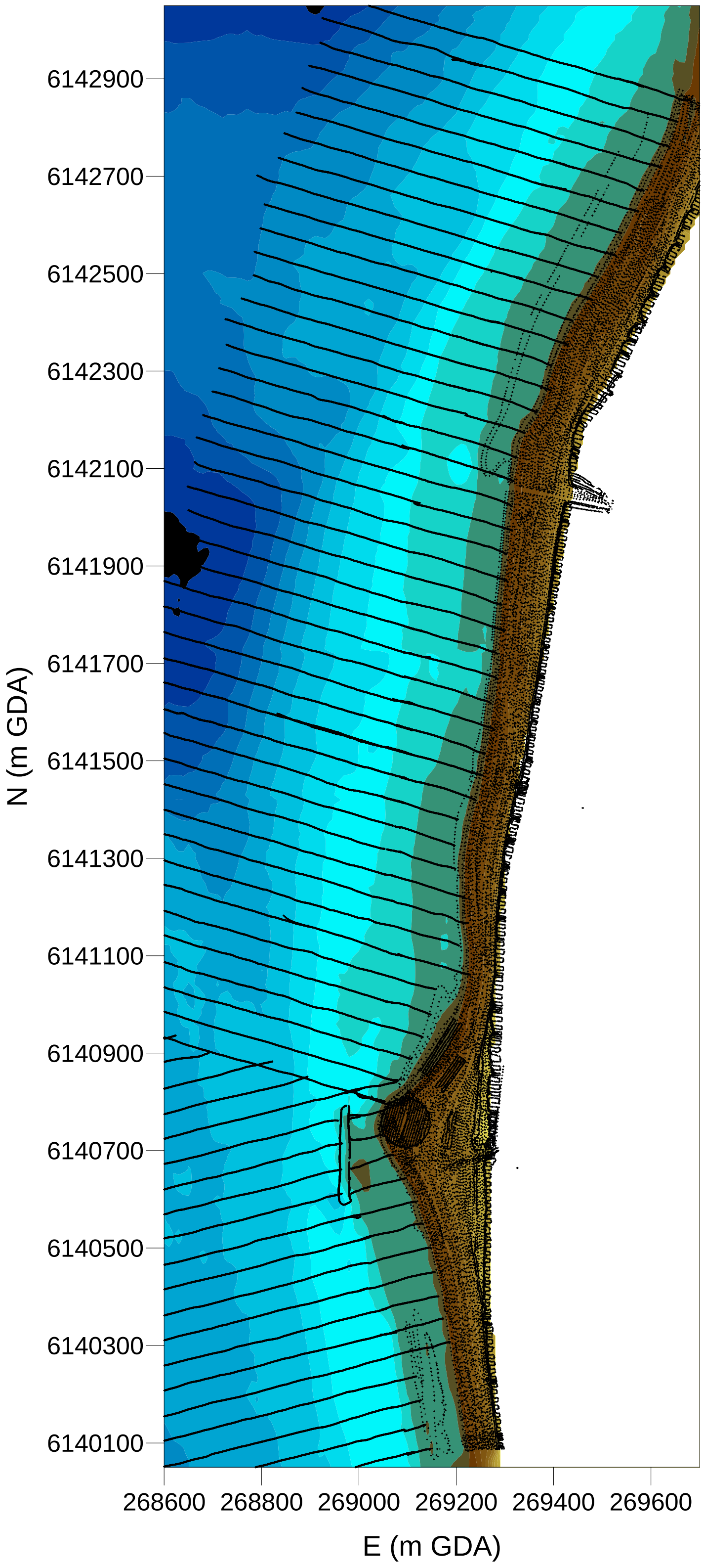


Cross Section

Variable	Symbol	Value
Crest length		200 m
Crest level		+1 m AHD
Toe level		-3 m AHD
Typical bed level		-2 m AHD
Scoured bed level		-2.5 m AHD
Rock density	ρ_{rock}	2700 kg/m^3
Porosity		0.37
Primary armour mass	M50	5 tonnes
Primary armour cube side equivalent	Dn50	1.3 m
Thickness of two layers (primary armour)		2.5 m
Minimum crest width		3.9 m
Side slope		1V:1.5H
Secondary armour	M50	500 kg
Secondary armour cube side equivalent	Dn50	0.6
Thickness of two layers (secondary)		1.1

Appendix A

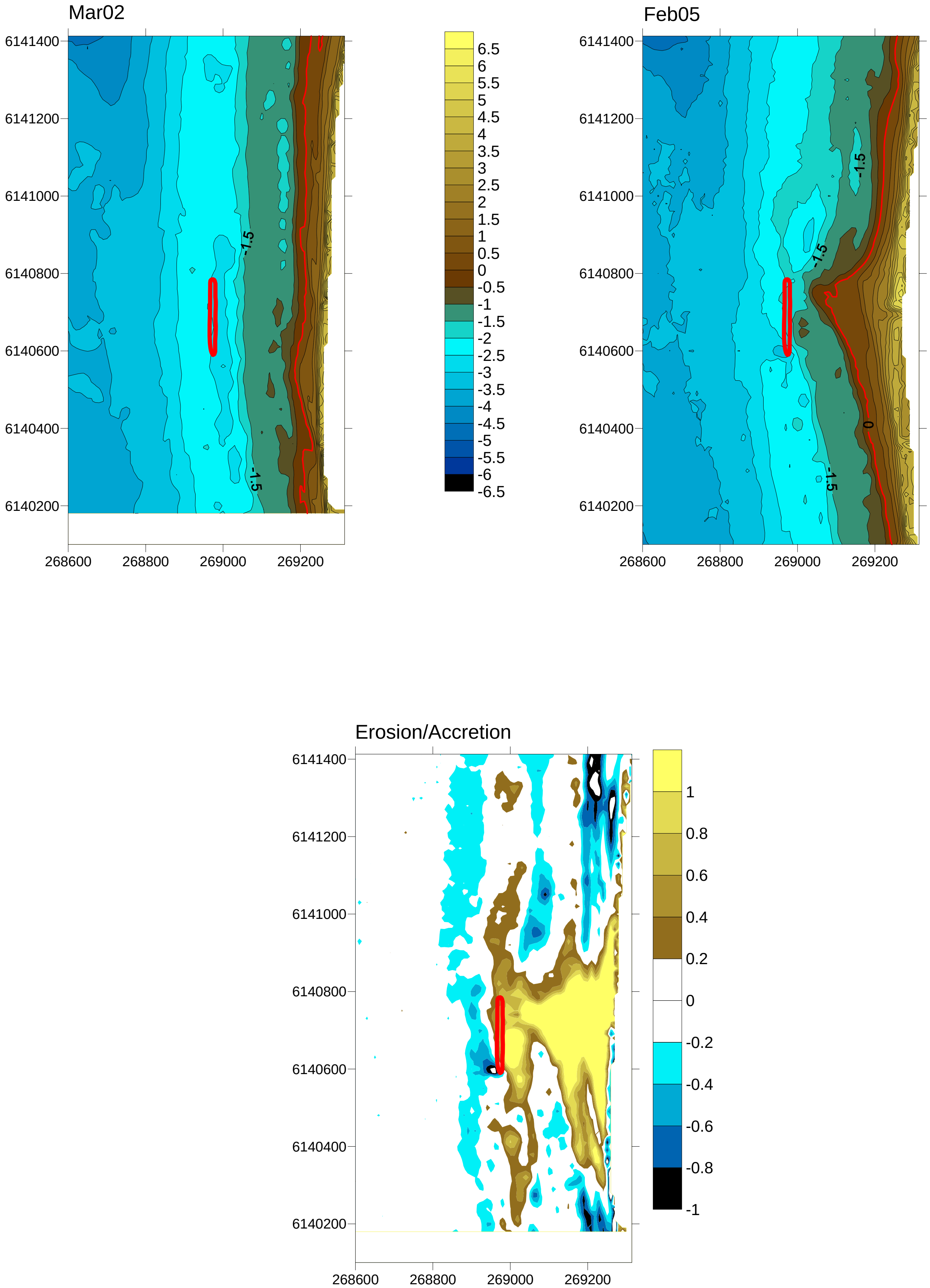
Survey Tracks



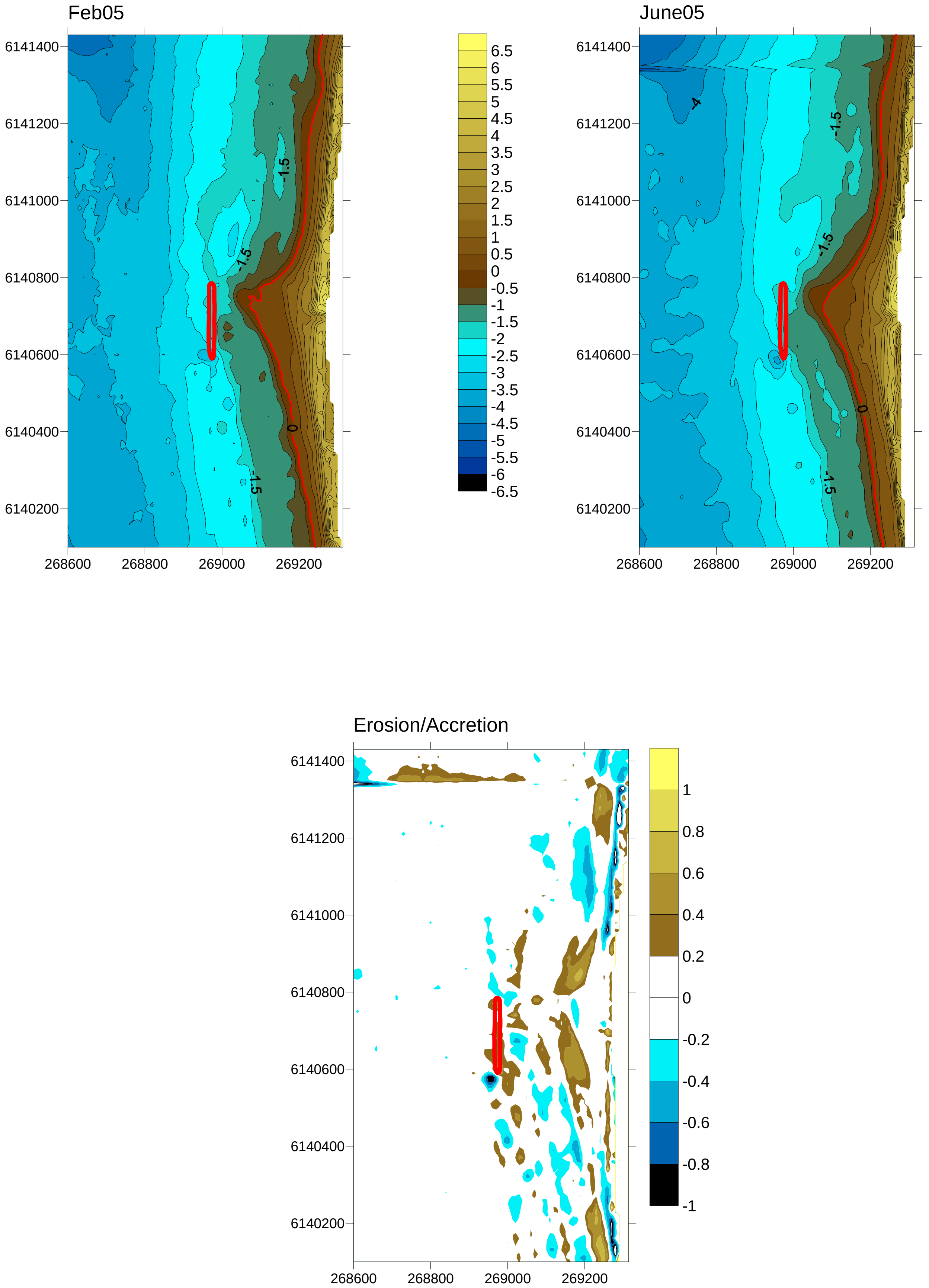
WRL Report No. 2007/14	XY SURVEY TRACKS	Figure A1

Appendix B

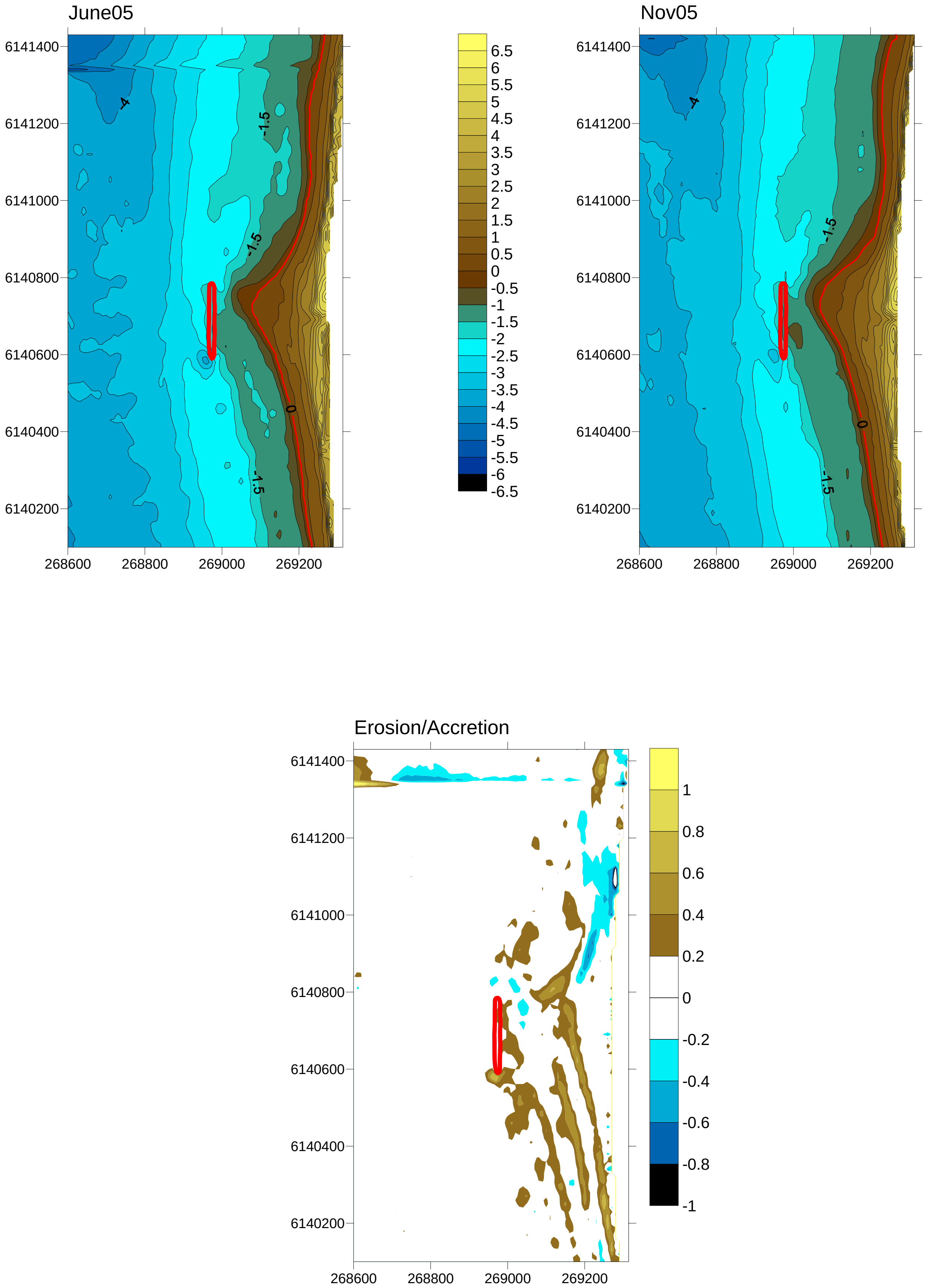
Contours and Isopachs in Breakwater Area



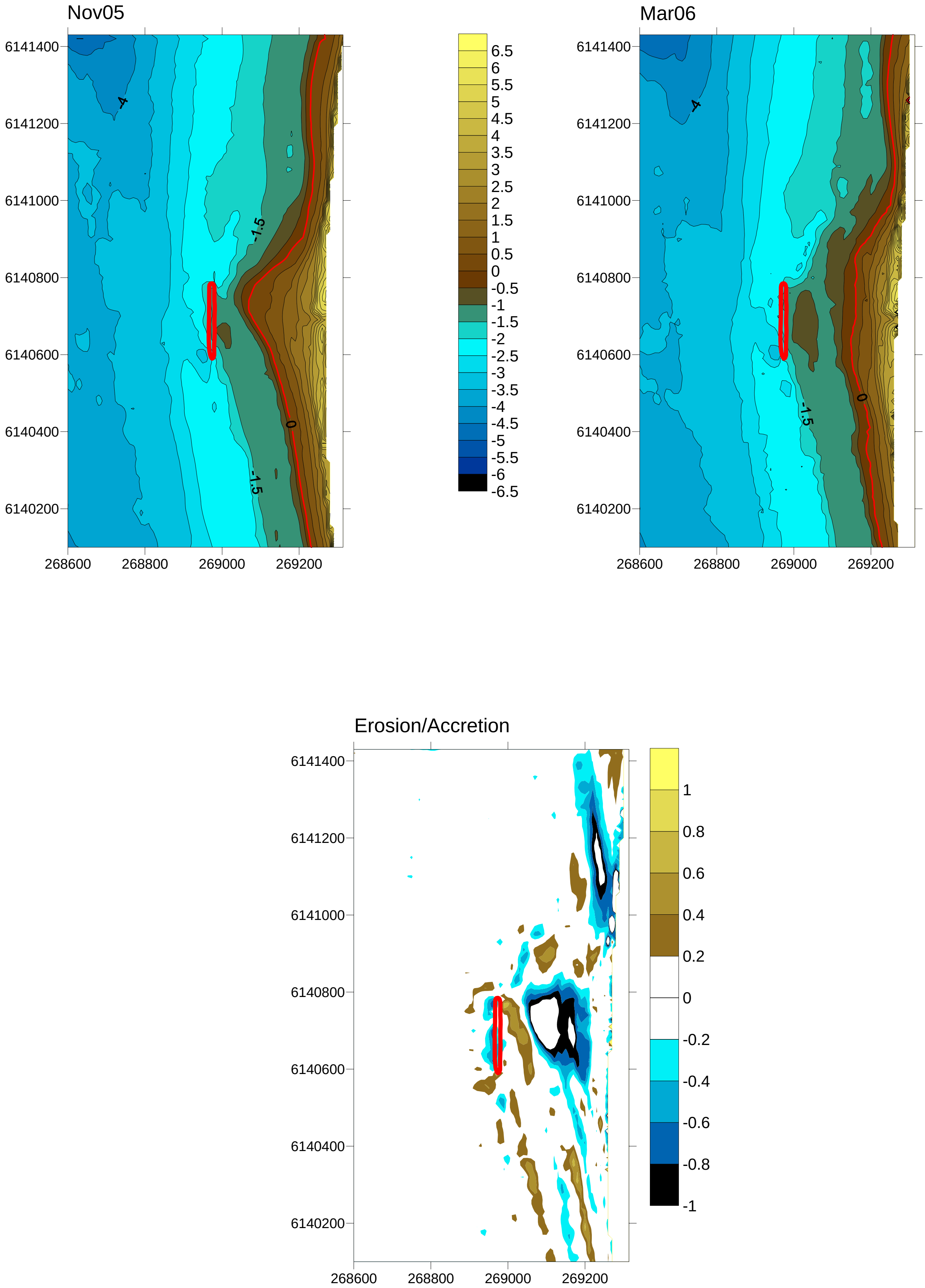
WRL Report No. 2007/14	EROSION / ACCRETION MARCH 02 – FEBRUARY 05	Figure B1



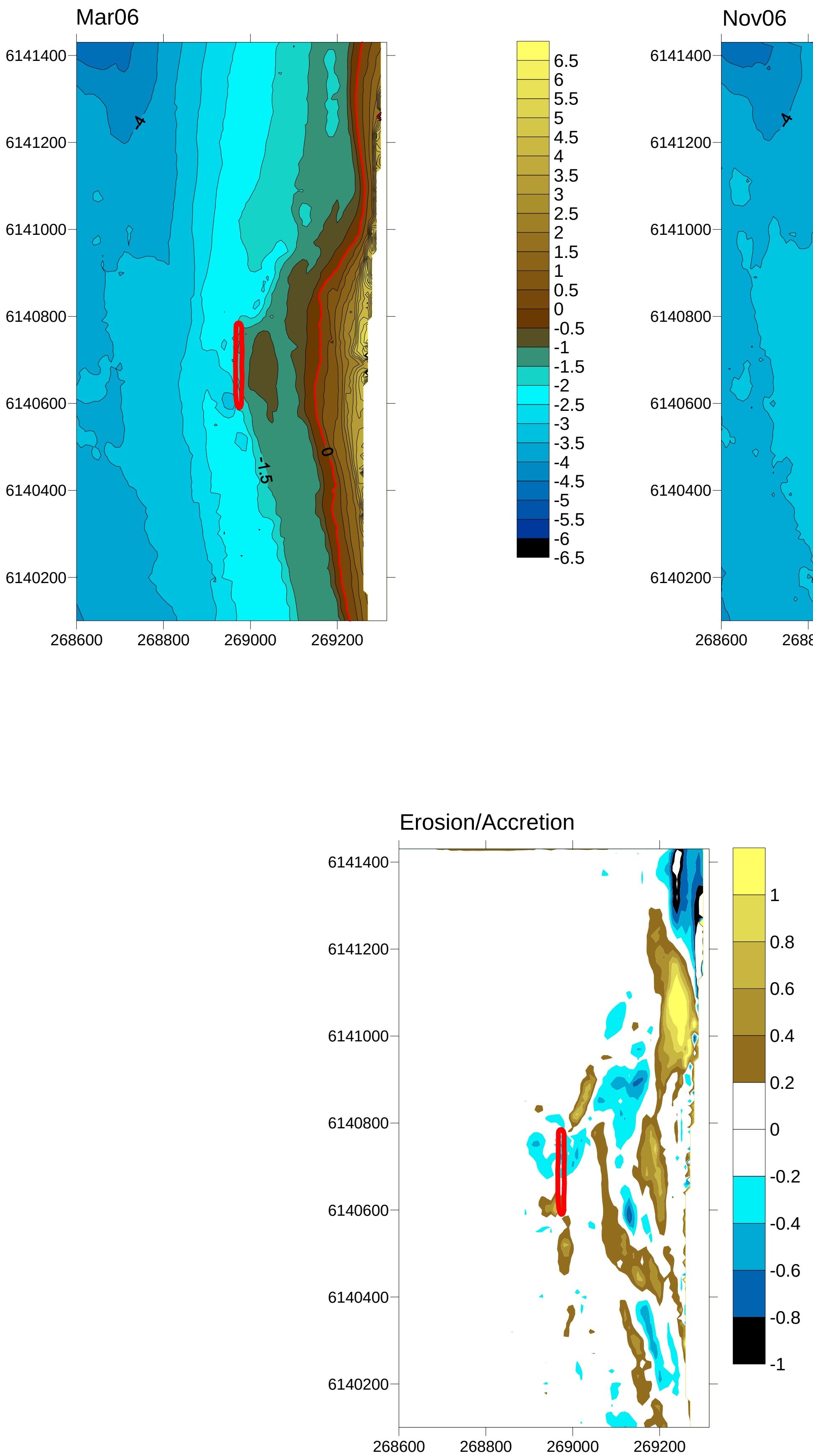
WRL Report No. 2007/14	EROSION / ACCRETION FEBRUARY 05 – JUNE 05	Figure B2



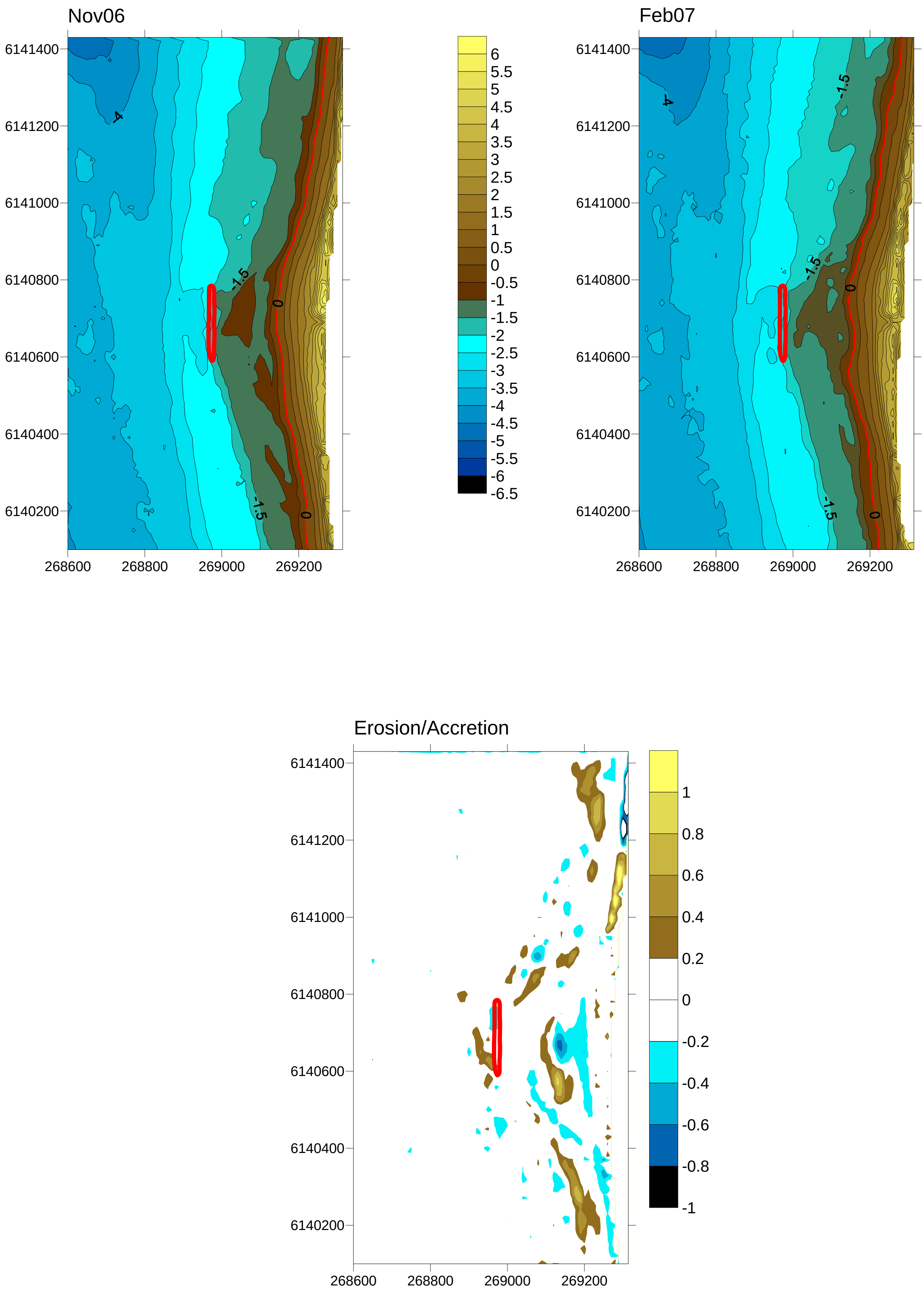
WRL Report No. 2007/14	EROSION / ACCRETION JUNE 05 – NOVEMBER 05	Figure B3



WRL Report No. 2007/14	EROSION / ACCRETION NOVEMBER 05 – MARCH 06	Figure B4



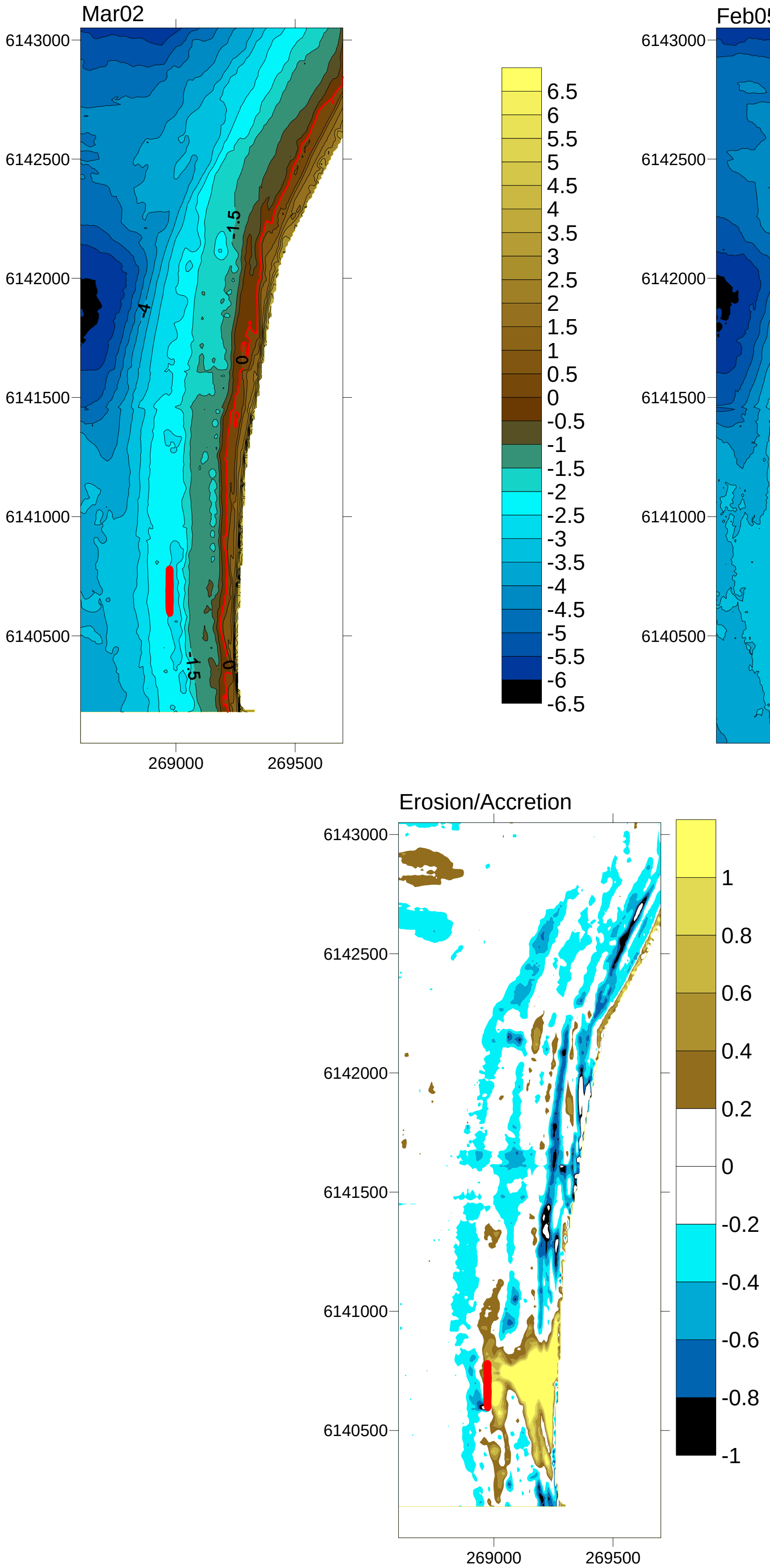
WRL Report No. 2007/14	EROSION / ACCRETION MARCH 06 – NOVEMBER 06	Figure B5



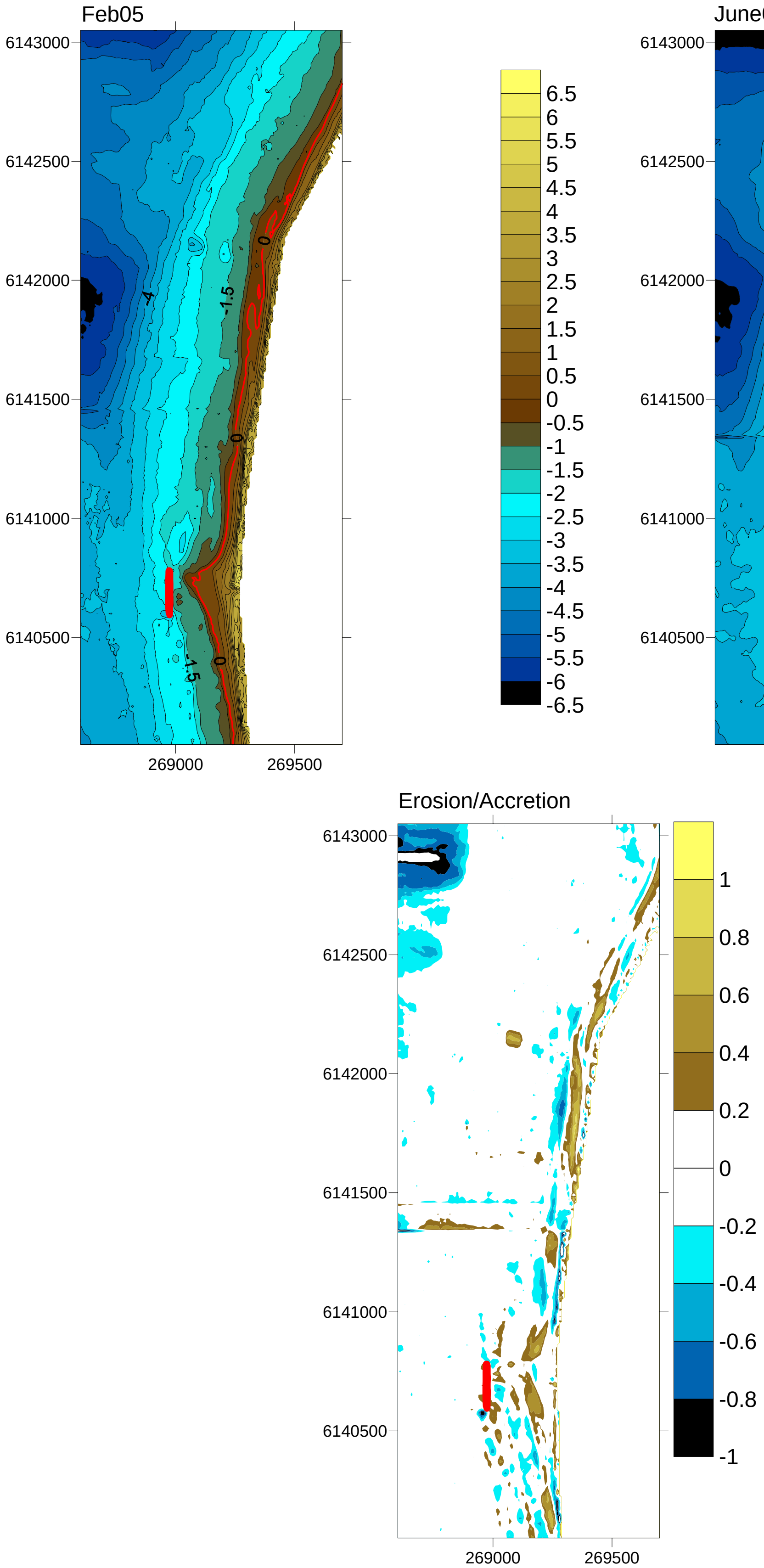
WRL Report No. 2007/14	EROSION / ACCRETION NOVEMBER 06 – FEBRUARY 07	Figure B6

Appendix C

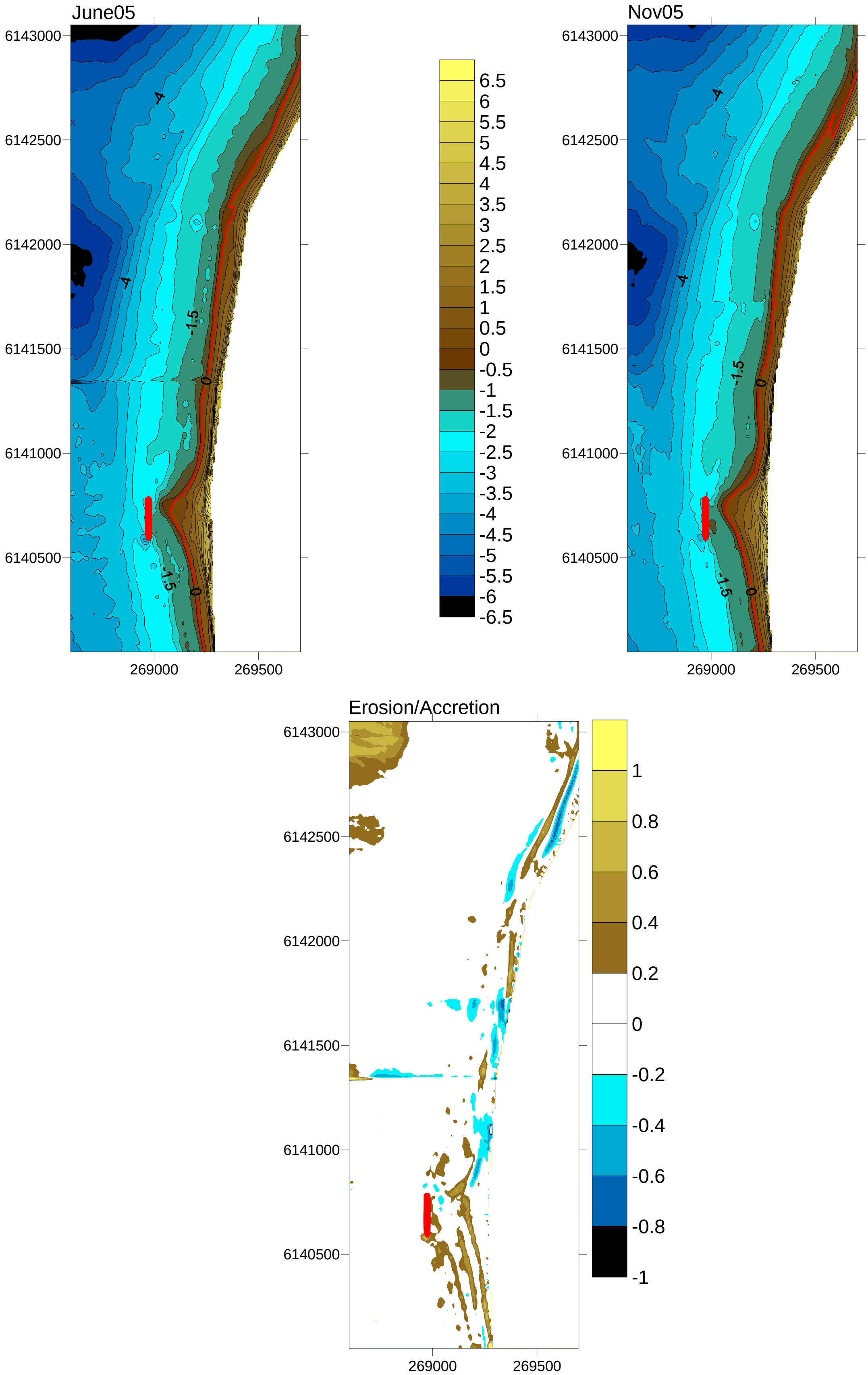
Contours and Isopachs in Full Survey Area



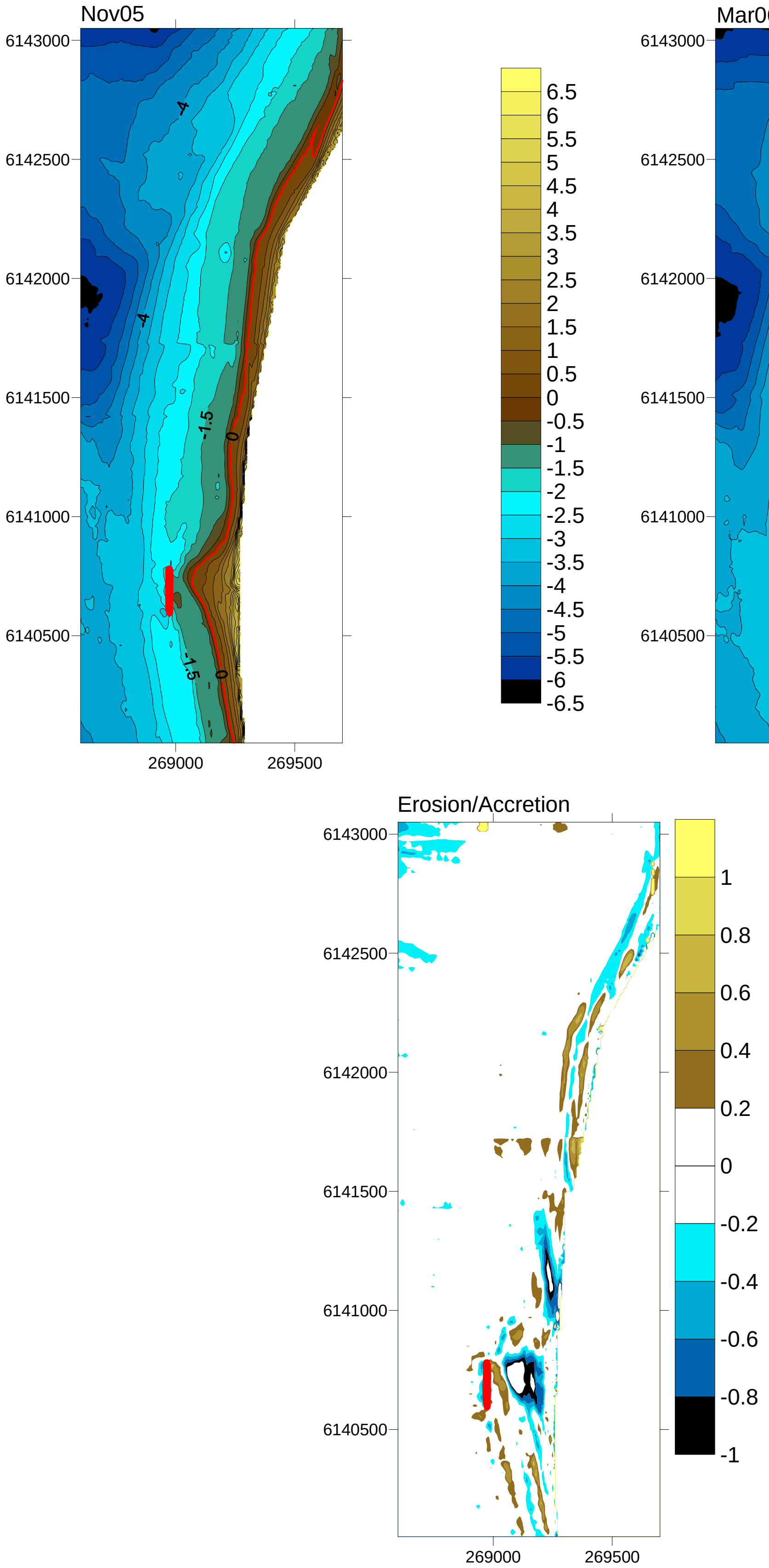
WRL Report No. 2007/14	NORTHERN EXTENT EROSION / ACCRETION MARCH 02 - FEBRUARY 05	Figure C1



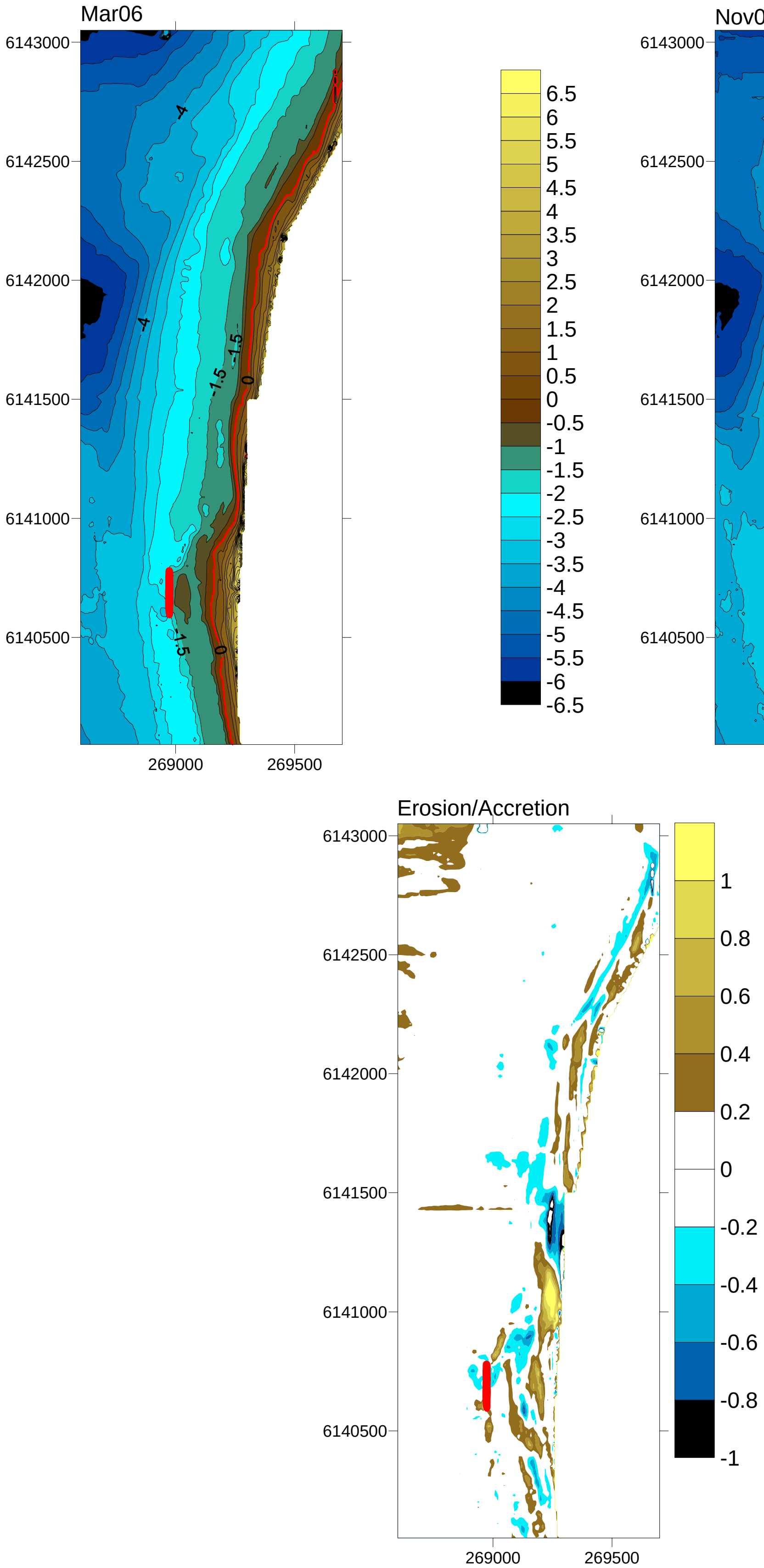
WRL Report No. 2007/14	NORTHERN EXTENT EROSION / ACCRETION FEBRUARY 05 - JUNE 05	Figure C2



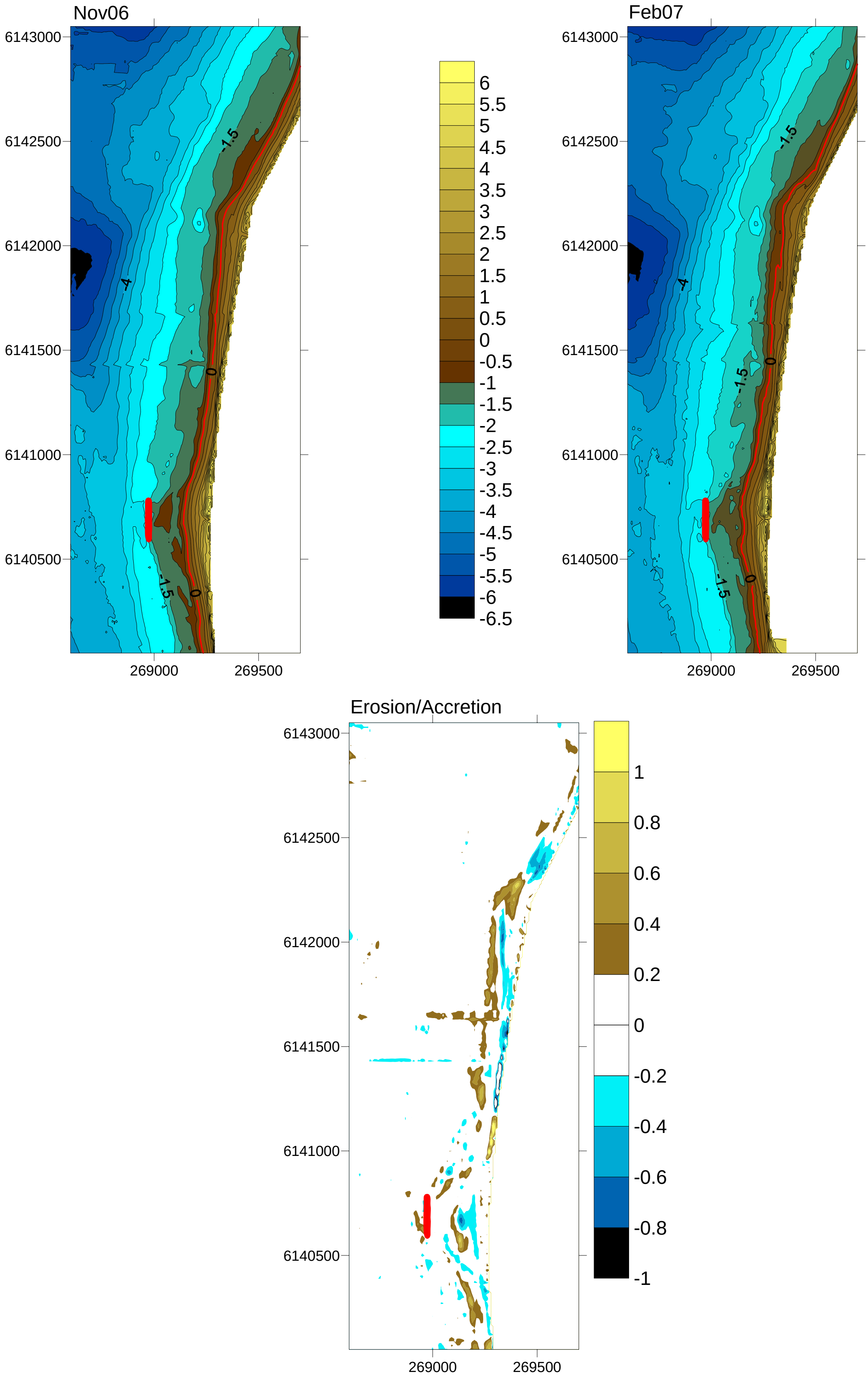
WRL Report No. 2007/14	NORTHERN EXTENT EROSION / ACCRETION JUNE 05 - NOVEMBER 05	Figure C3



WRL Report No. 2007/14	NORTHERN EXTENT EROSION / ACCRETION NOVEMBER 05 - MARCH 06	Figure C4



WRL Report No. 2007/14	NORTHERN EXTENT EROSION / ACCRETION MARCH 06 – NOVEMBER 06	Figure C5



WRL Report No. 2007/14	NORTHERN EXTENT EROSION / ACCRETION NOVEMBER 06 – FEBRUARY 07	Figure C6

Appendix D

Satellite Photos

Semaphore Park
Satellite Imagery
Date 31-3-2002 (Time 00:54:21 UTC)

Semaphore Park
Satellite Imagery
Date 25-10-2004 (Time 00:56:34 UTC)

Semaphore Park
Satellite Imagery
Date 18-12-2004 (Time 00:59:54 UTC)

Semaphore Park
Satellite Imagery
Date 23-2-2005 (Time 00:58:45 UTC)

Semaphore Park
Satellite Imagery
Date 6-5-2005 (Time 00:03:07 UTC)

Semaphore Park
Satellite Imagery
Date 22-8-2005 (Time 01:08:56 UTC)

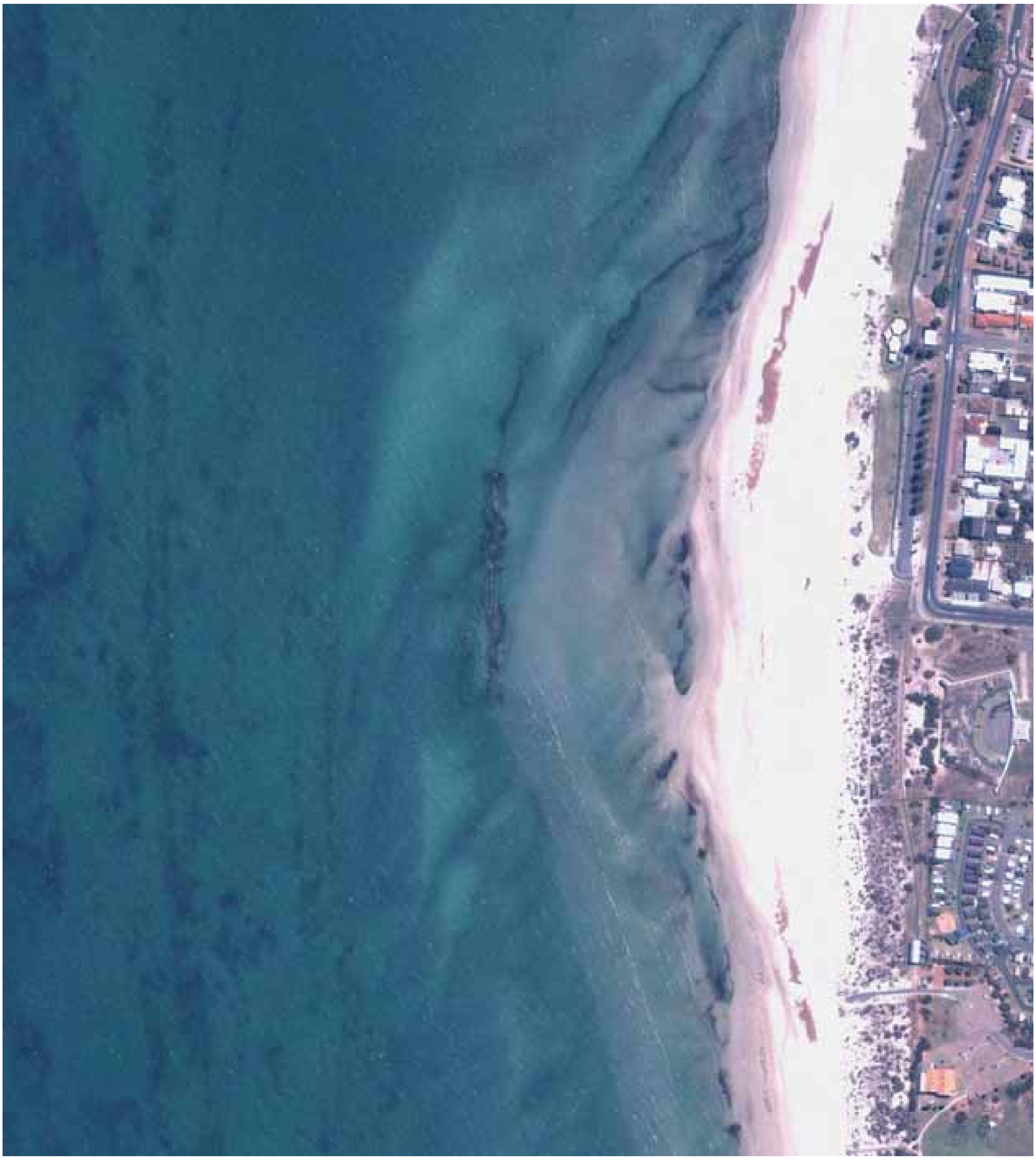
Semaphore Park
Satellite Imagery
Date 26-4-2006 (Time 01:14:47 UTC)

Semaphore Park
Satellite Imagery
Date 30-7-2006 (Time 01:23:16 UTC)

Semaphore Park
Satellite Imagery
Date 30-9-2006 (Time 01:15:26 UTC)

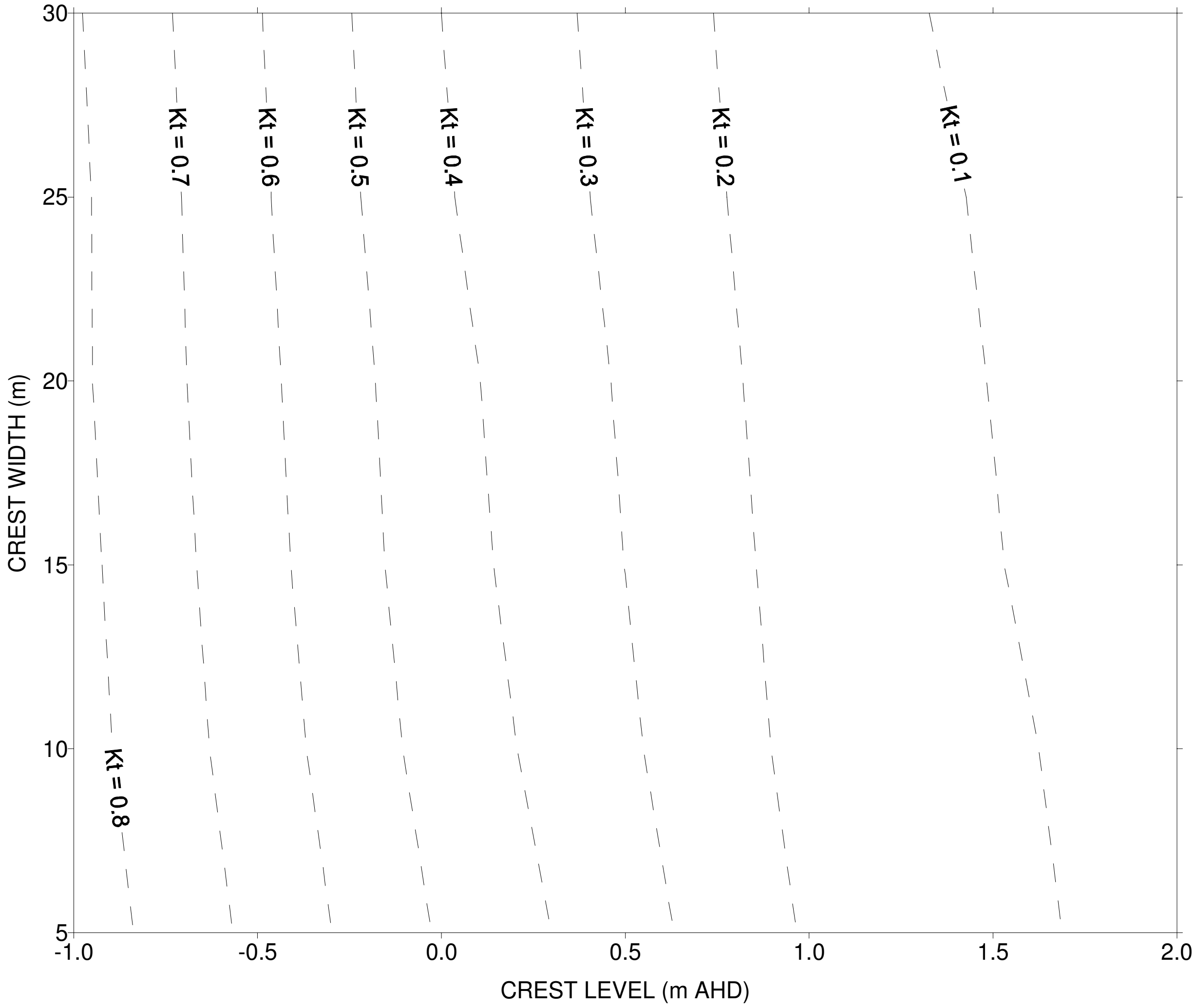
Semaphore Park
Satellite Imagery
Date 28-11-2006 (Time 01:21:34 UTC)

Semaphore Park
Satellite Imagery
Date 29-01-2007



Appendix E

Figures from WRL Technical Report 01/24 Relevant to this Study

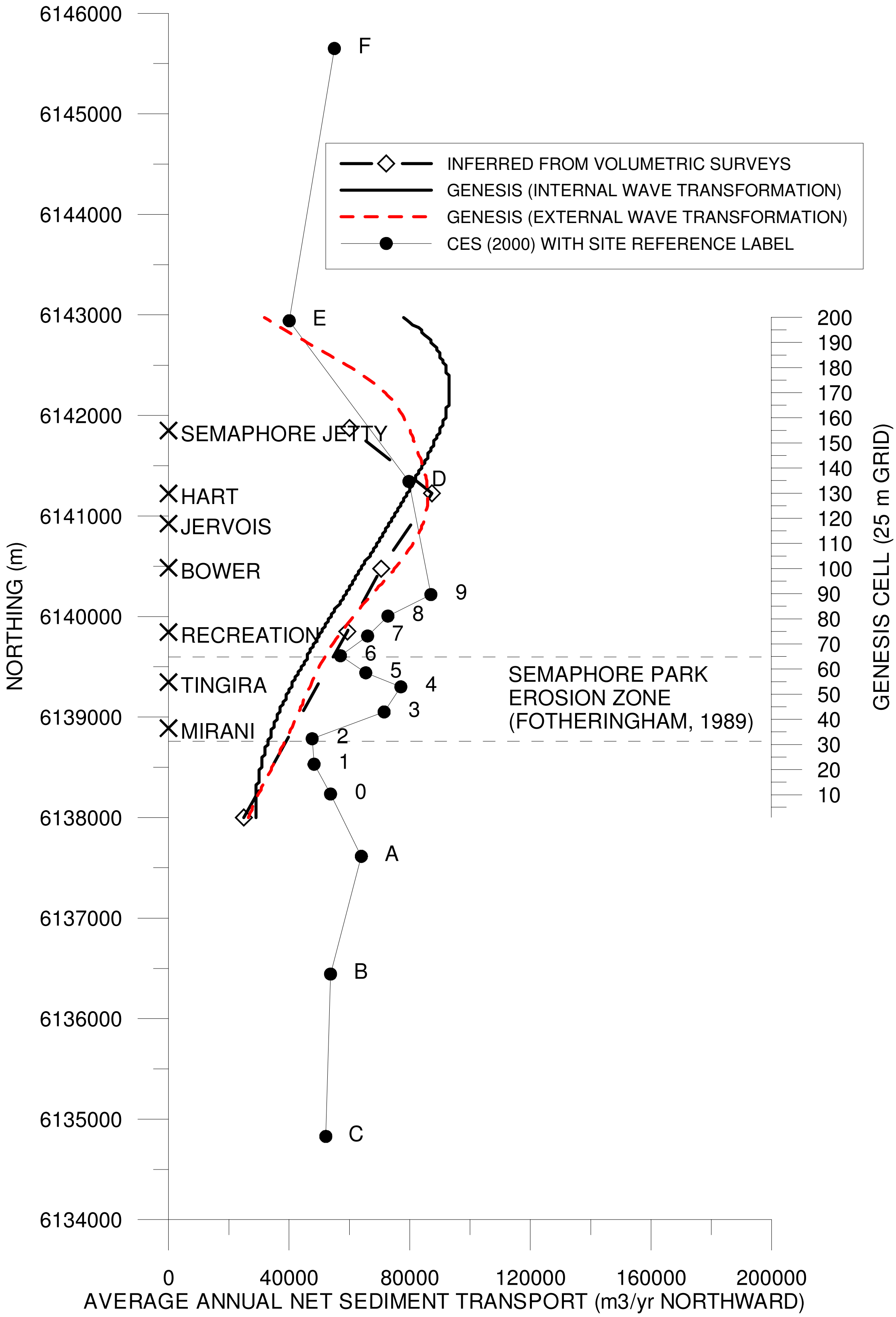


Based on:

- Toe level of –2 m AHD
- Side slopes of 1:2
- 6 years of hindcast sea data in CES (2000)

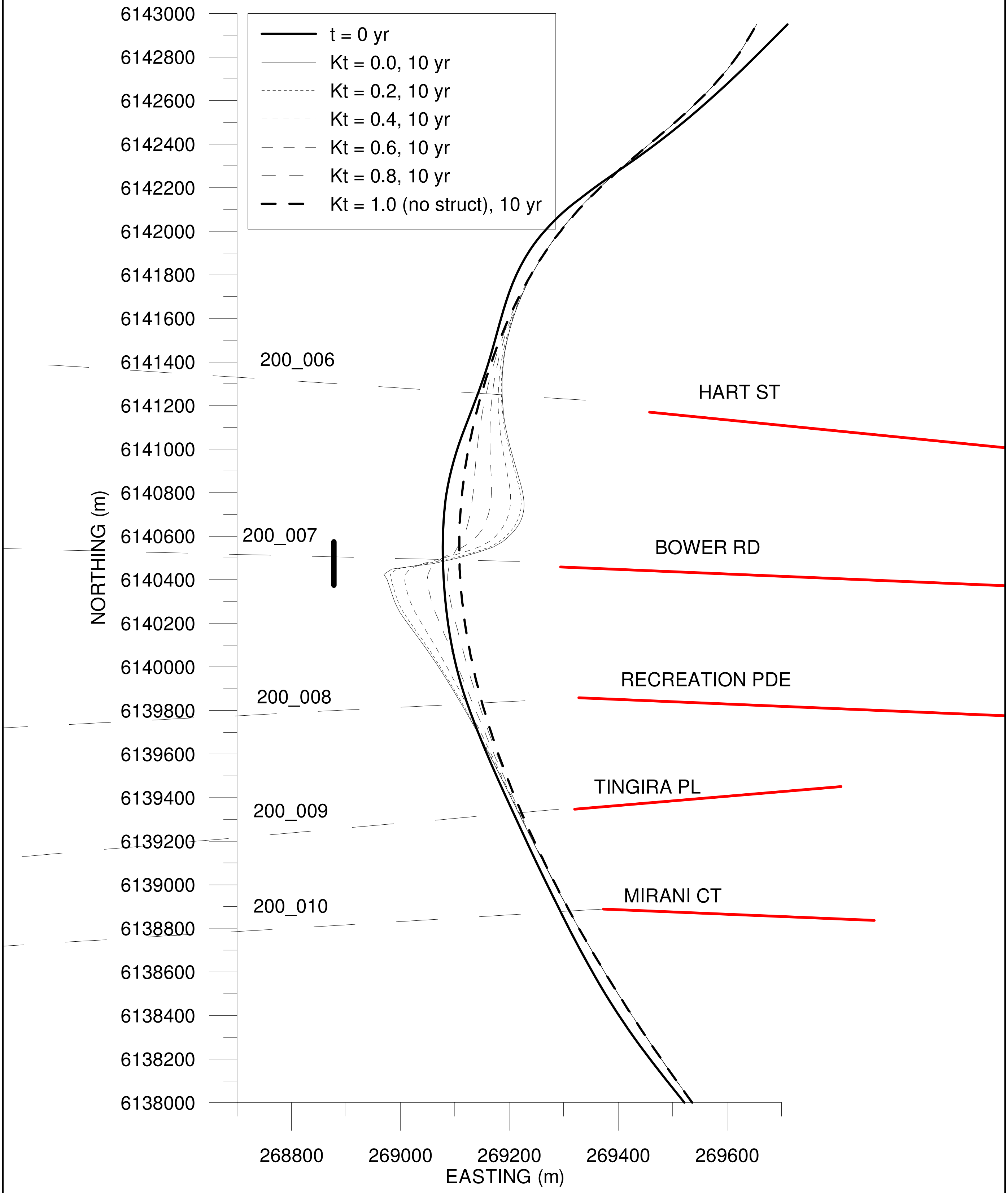
WRL recommendation Kt av < 0.4

WRL Report No. 01/24	AVERAGE (SEA) WAVE TRANSMISSION COEFFICIENT AS A FUNCTION OF CREST LEVEL AND WIDTH	Figure 6.3 01007-6-3



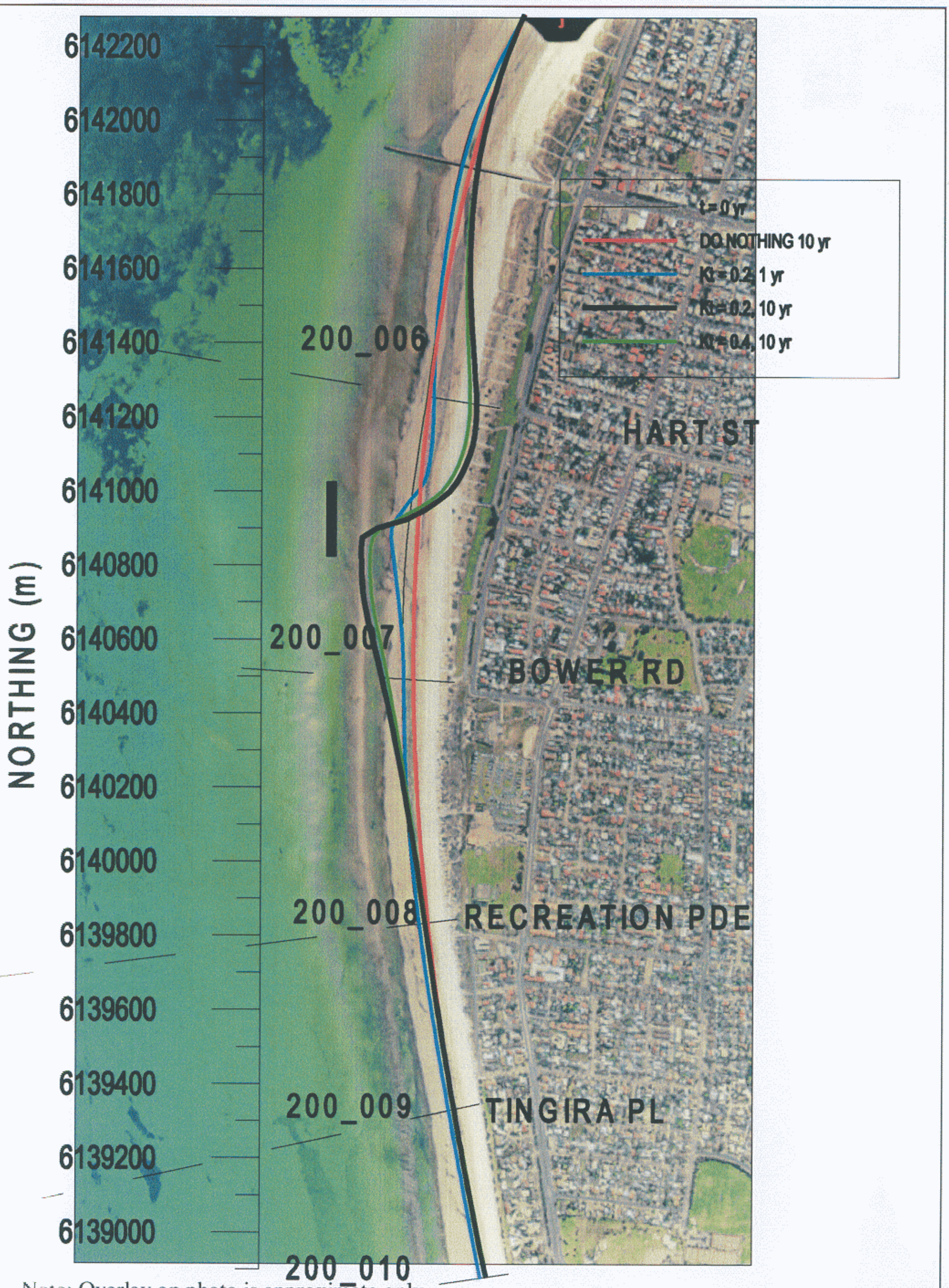
Note: On the basis of net northward transport, areas where the line slopes up to the right would erode, areas where the line slopes up to the left would accrete

WRL Report No. 01/24	COMPARISON OF CALCULATED SEDIMENT TRANSPORT RATES	Figure 8.2
		01007-8-2



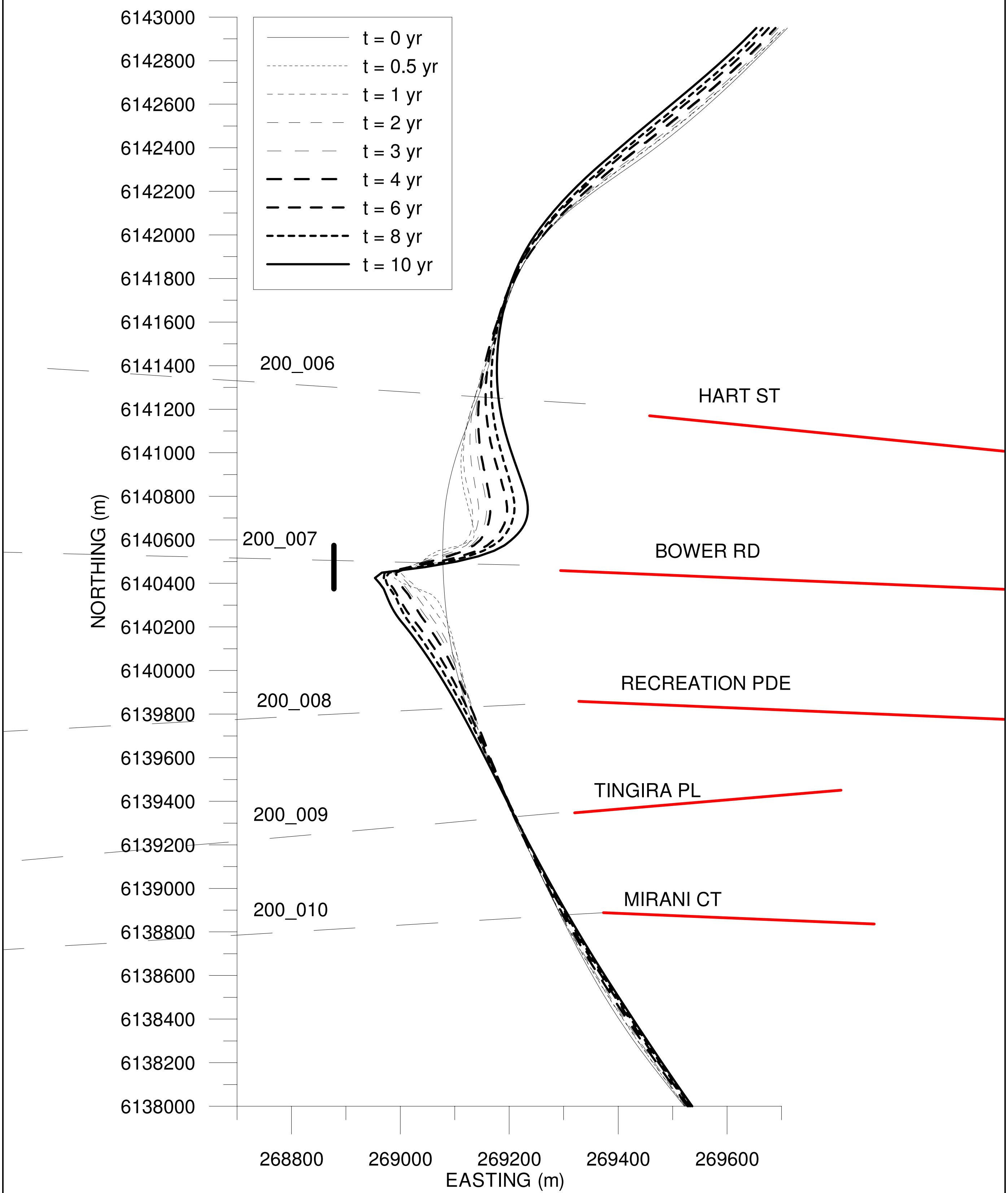
Note: Easting to northing is distorted by factor of 2.5

WRL Report No. 01/24	GENESIS SCOPING MODELLING, BOWER ROAD, EFFECT OF Kt B = 200 m, S = 200	Figure 8.16 01007-8-16



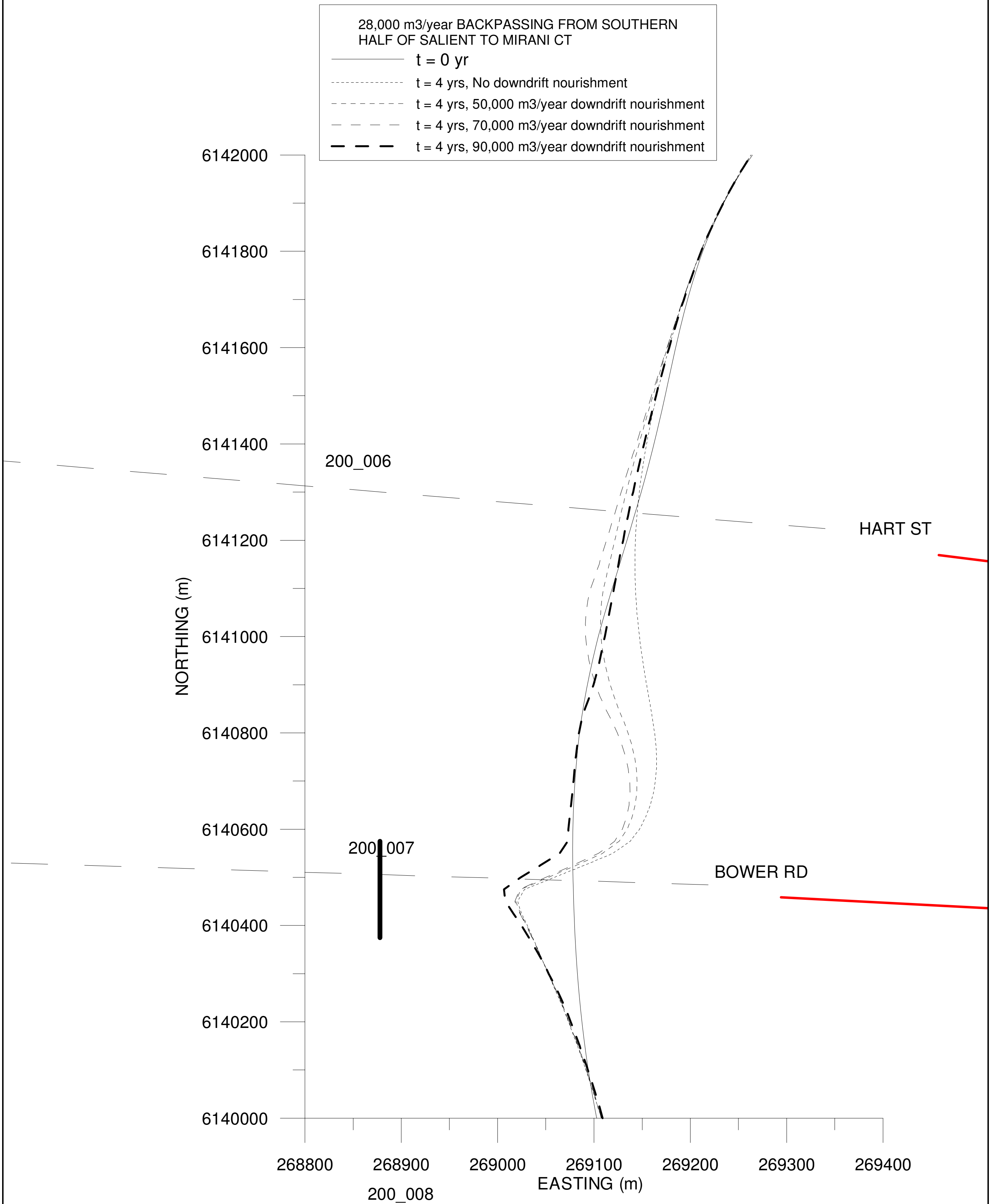
Note: Overlay on photo is approximate only

WRL Report No. 01/24	GENESIS SCOPING MODELLING, JERVOIS STREET, PHOTO OVERLAY, NO NOURISHMENT, B = 200 m, S = 200	Figure 8.22 01007-8-22.DWG



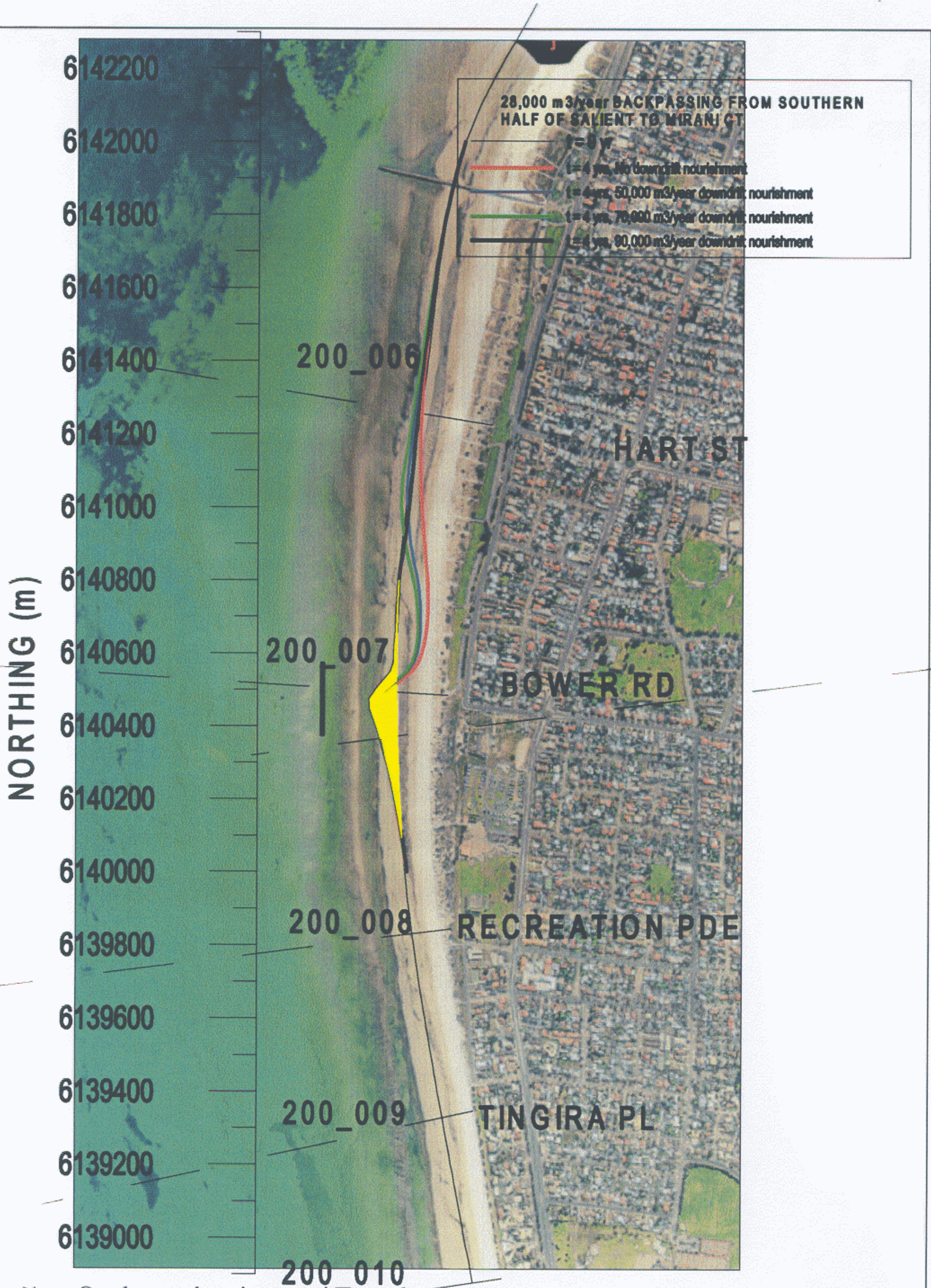
Note: Easting to northing is distorted by factor of 2.5

WRL Report No. 01/24	GENESIS DETAILED DESIGN MODELLING, BOWER RD B = 200 m, S = 200, Kt = 0.2	Figure 8.24 01007-8-24



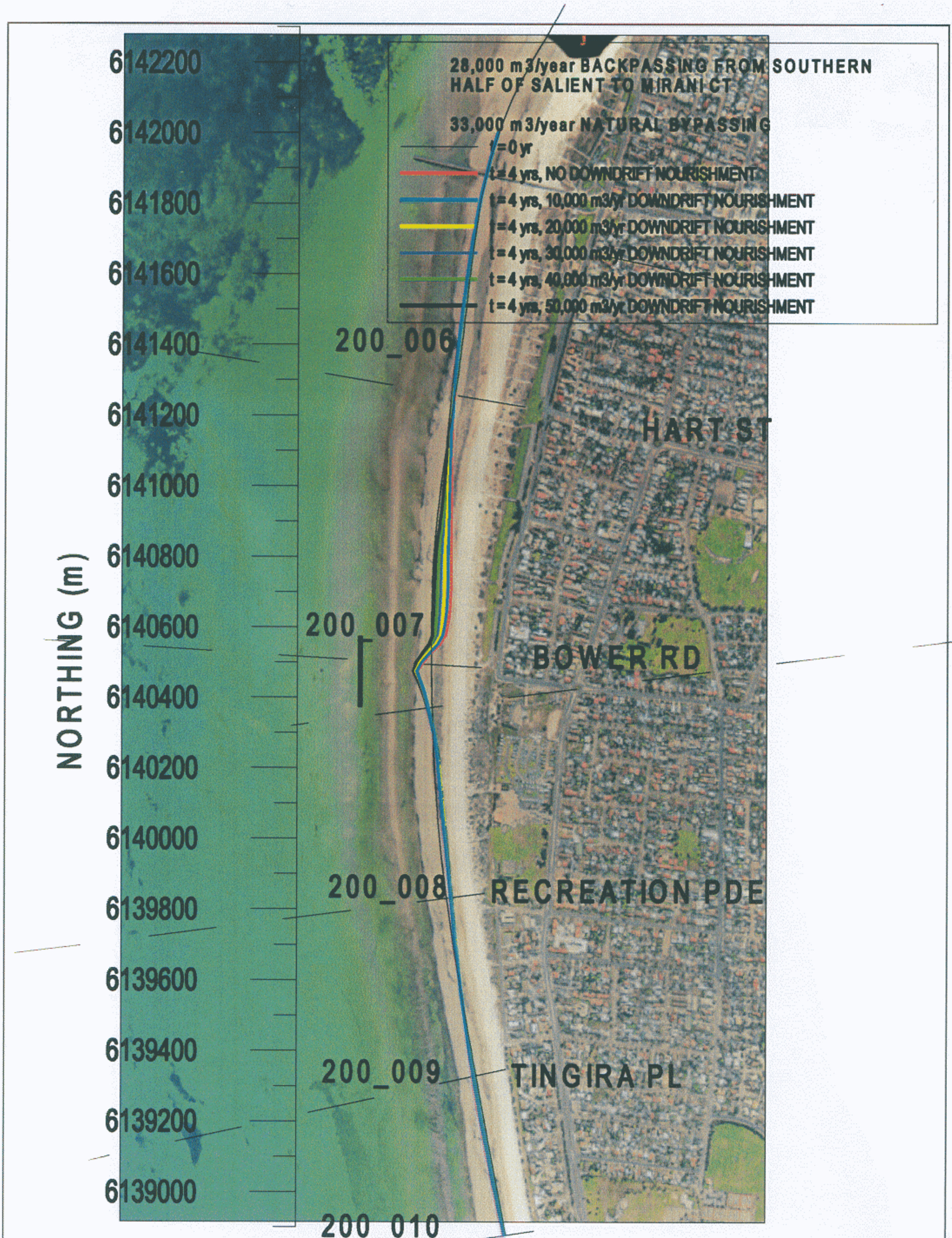
Note: Easting to northing is distorted by factor of 2

WRL Report No. 01/24	GENESIS DETAILED DESIGN MODELLING, BOWER ROAD, BACKPASSING AND DOWNDRIFT NOURISHMENT B = 200 m, S = 200, Kt = 0.2	Figure 8.29 01007-8-29



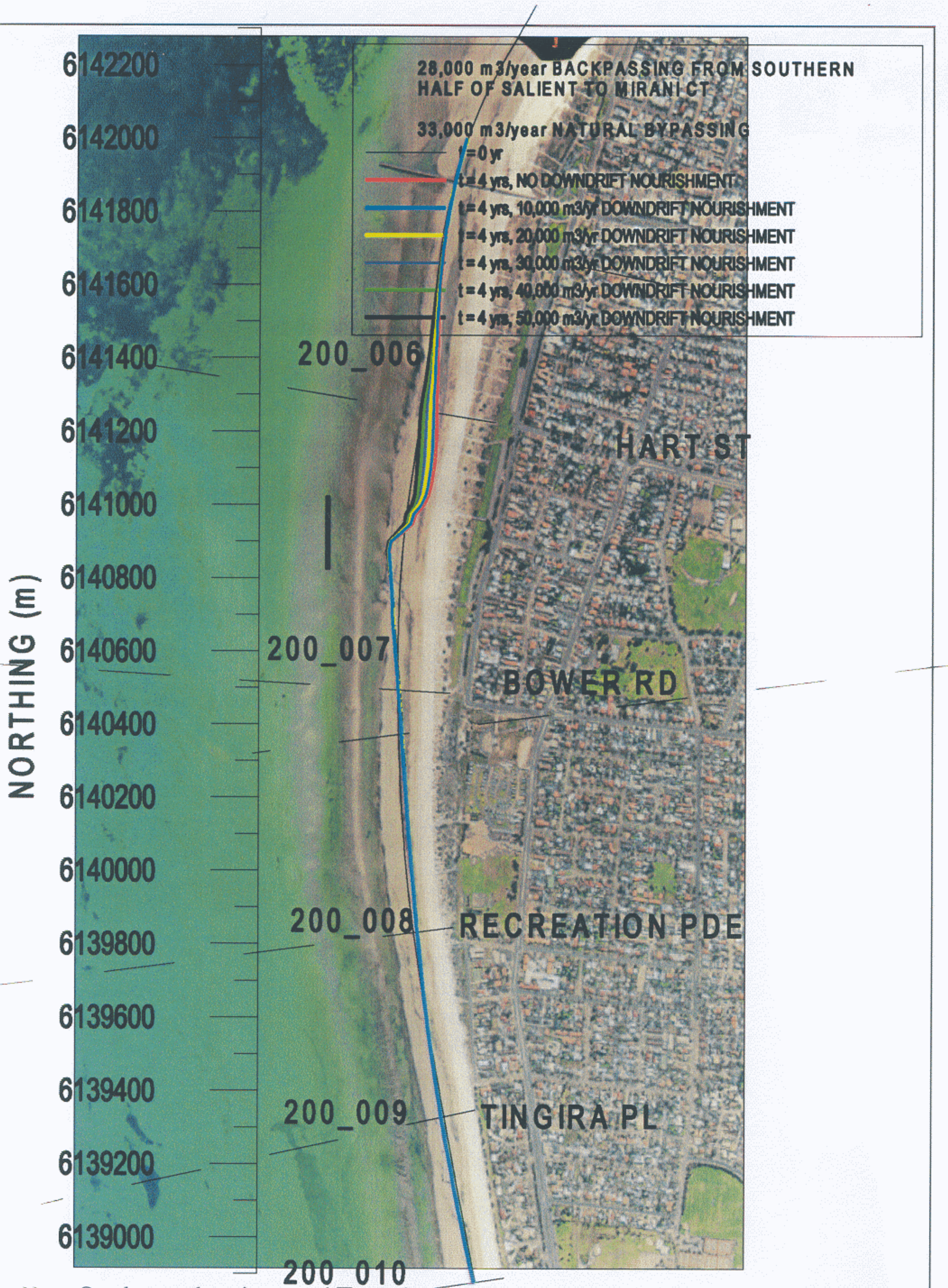
Note: Overlay on photo is approximate only

WRL Report No. 01/24	GENESIS DETAILED DESIGN MODELLING, BOWER ROAD, PHOTO OVERLAY, BACKPASSING AND DOWNDRIFT NOURISHMENT, B = 200 m, S = 200, Kt = 0.2	Figure 8.31 01007-8-31.DWG



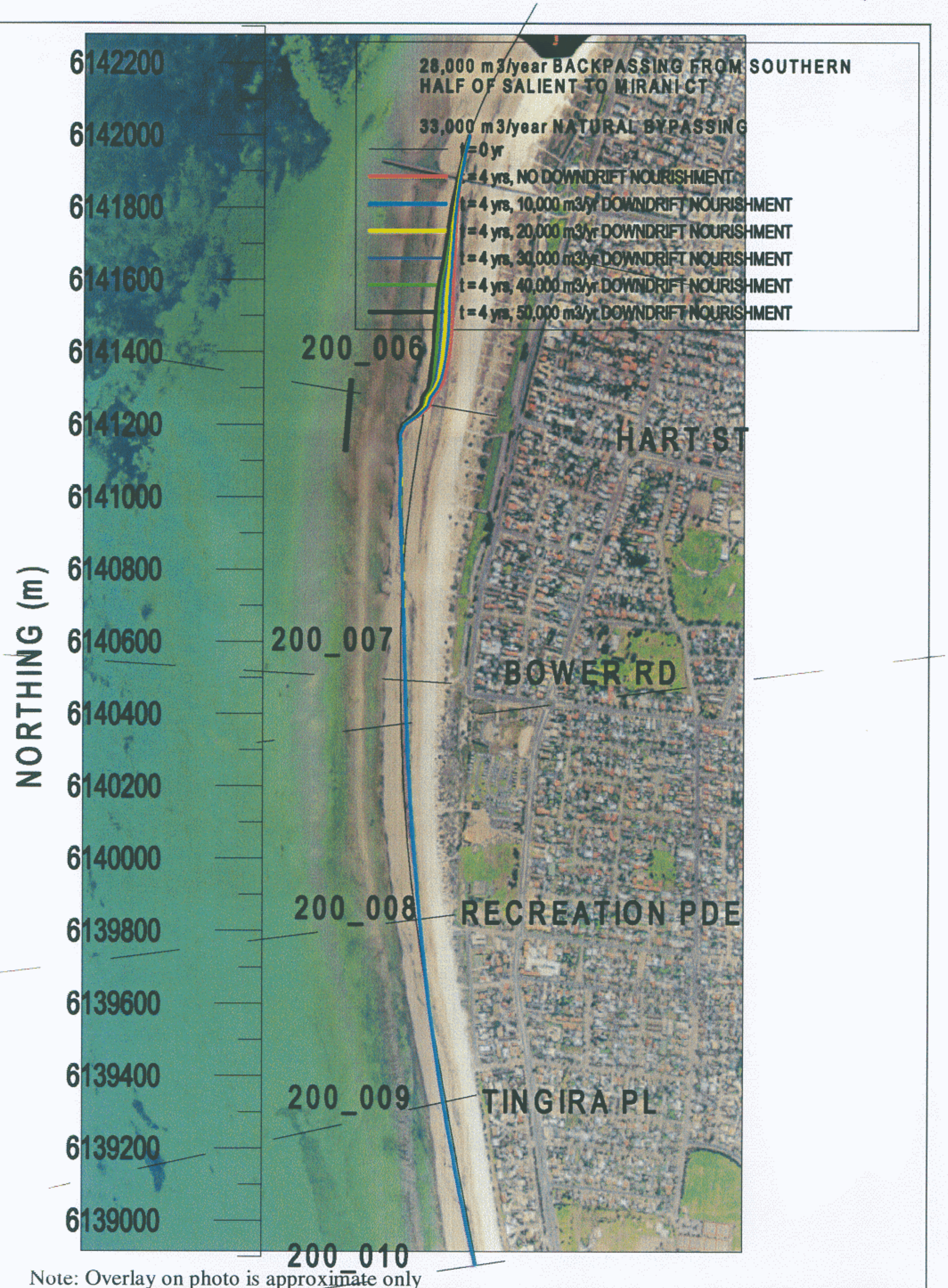
Note: Overlay on photo is approximate only

WRL Report No. 01/24	GENESIS DETAILED MODELLING, BOWER RD, PHOTO OVERLAY, 28,000 m^3/yr BACKPASSING, 33,000 m^3/yr NATURAL BYPASSING, DOWNDRIFT NOURISHMENT, B=200 m, S=200, Kt=0.2	Figure 8.36 01007-8-36.DWG



Note: Overlay on photo is approximate only

WRL Report No. 01/24	GENESIS DETAILED MODELLING, JERVOIS, PHOTO OVERLAY, 28,000 m^3/yr BACKPASSING, 33,000 m^3/yr NATURAL BYPASSING, DOWNDRIFT NOURISHMENT, B=200 m, S=200, Kt=0.2	Figure 8.37 01007-8-37.DWG



Note: Overlay on photo is approximate only

WRL Report No. 01/24	GENESIS DETAILED MODELLING, HART ST, PHOTO OVERLAY, 28,000 m^3/yr BACKPASSING, 33,000 m^3/yr NATURAL BYPASSING, DOWNDRIFT NOURISHMENT, B=200 m, S=200, Kt=0.2	Figure 8.38 01007-8-38.DWG