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Design Criteria Report for Board Walk

Witton Bluff, Port Noarlunga, SA

Onkaparinga City Council

15 September 2020

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

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GLOSSARY AND DEFINITIONS

AHD	Australian Height Datum. A common national surface level datum approximately corresponding to mean sea level.
ARI	Average Recurrence Interval. The average or expected value of the periods between exceedances of a given event.
CSIRO	The Commonwealth Scientific and Industrial Research Organisation, the federal government agency for scientific research in Australia.
CD	Chart Datum – LAT in this study.
HAT	Highest Astronomical Tide
$H_{1\%}$	Defined as the average of the highest 1% of waves
H_b	Breaking wave height.
H_{max}	Maximum wave height
H_s	Significant wave height. Defined as the mean wave height (trough to crest) of the highest third of the waves during a given period.
IPCC	Intergovernmental Panel on Climate Change. A scientific and intergovernmental body under the auspices of the United Nations, set up at the request of member governments, dedicated to the task of providing the world with an objective, scientific view of climate change and its political and economic impacts.
LAT	Lowest Astronomical Tide
MHWN	Mean High Water Neap.
MHWS	Mean High Water Springs
MLWN	Mean Low Water Neap
MLWS	Mean Low Water Springs
MSL	Mean Sea Level
MSLR	Mean Sea Level Rise
Semi Diurnal Tides	Semidiurnal tide refers to a tide which has a period or cycle of approximately half of one tidal day (about 12.5 hours). Semidiurnal tides usually have two high and two low tides each day.
Storm Surge	The meteorological component of the coastal water level variations associated with atmospheric pressure fluctuations and wind setup.
Storm Tide	Coastal water level produced by the combination of astronomical and meteorological (storm surge) ocean water level forcing.
Tidal Planes	A series of water levels that define standard tides, eg. 'Mean High Water Spring' (MHWS) refers to the average high-water level of Spring Tides.
T	Wave period
T_p	Wave energy spectral peal wave period - that is, the wave period related to the highest ordinate in the wave energy spectrum.
T_z	Zero Crossing Wave Period. The average period of waves in a train of waves observed at a location.
L	Wave Length or the distance between two wave crests.

1 INTRODUCTION

1.1 Background

Onkaparinga City Council (OCC) received state government funding to construct a 3m wide shared use path around the base of Witton Bluff. The coastal path will extend around the base of the cliffs from Beach Road Christies to the Esplanade/Salt fleet Street intersection at Port Noarlunga (opposite the jetty). The works consist of widening of the existing path from Christies Beach to Cap Stephanie (Benny Avenue) and an elevated boardwalk structure from Cap Stephanie to Port Noarlunga Jetty (see Figure 1-1). This Design Criteria Report is developed for the elevated boardwalk section.

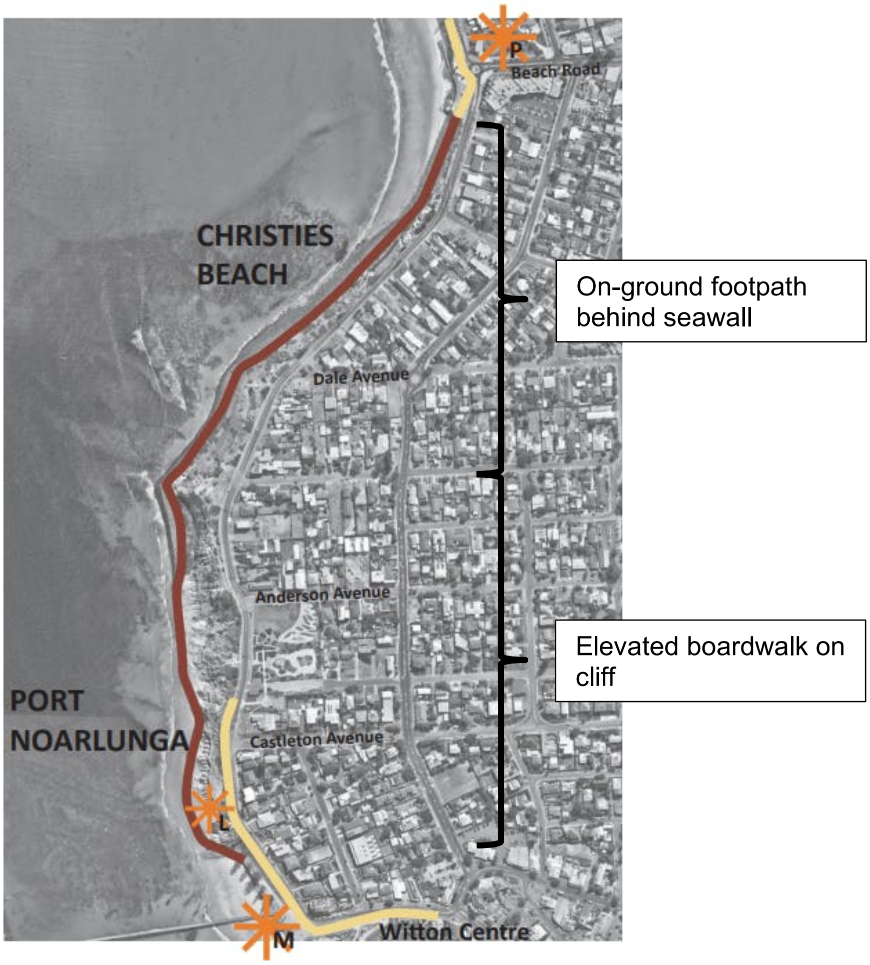

FIGURE 1-1 SHARED USE PATH ALIGNMENT

The elevated boardwalk will be supported by pile foundation as shown in Figure 1-2. Per concept drawings (Appendix A), most piles will be located on the cliff thereby subject to very limited wave load. There are however two segments where the boardwalk piles are exposed to significant wave impacts. The purpose of this report is to list the basis of design criteria and to evaluate the wave load impacts to the proposed boardwalk.

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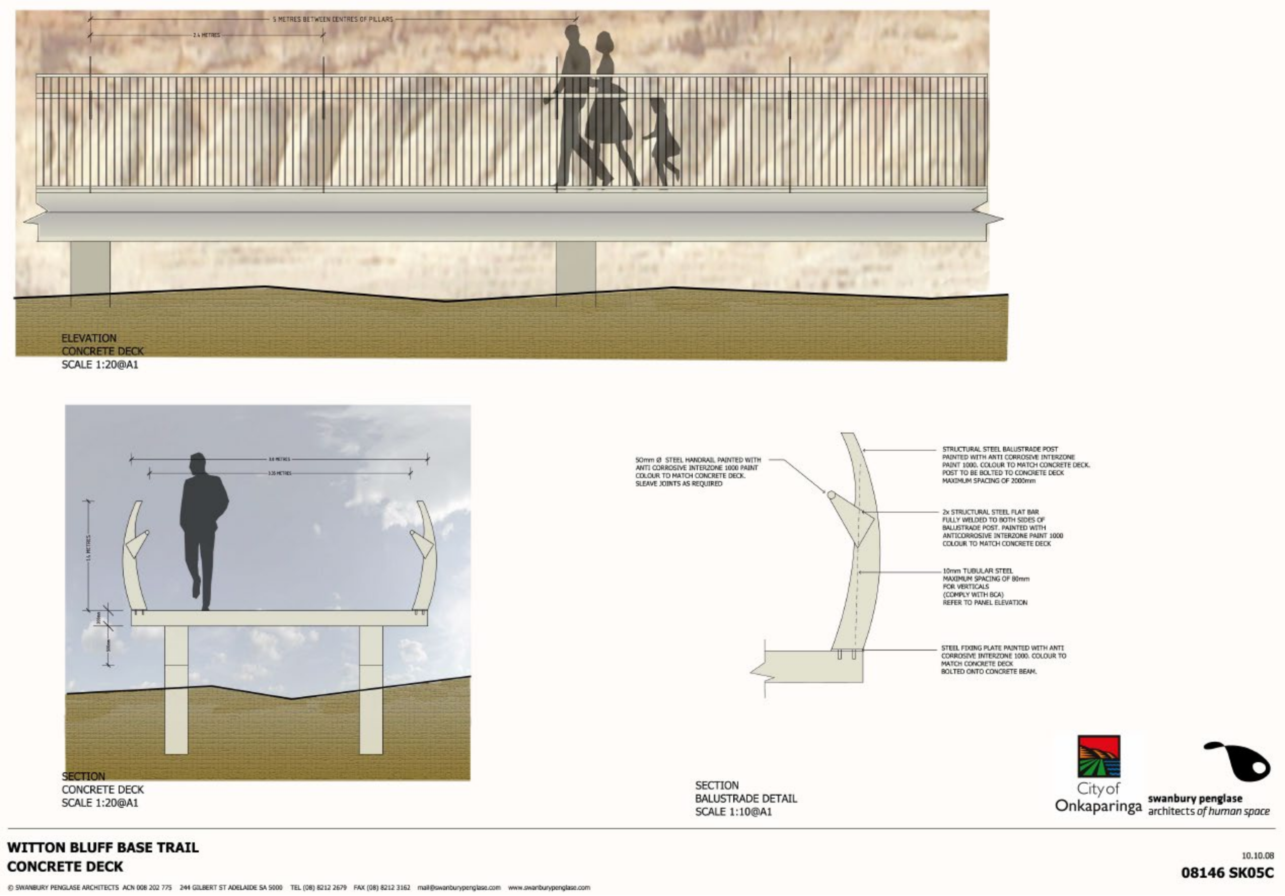

FIGURE 1-2 ELEVATED BOARDWALK CONCEPT DESIGN (WBBT CONCEPT DESIGN, 2008).

1.2 Scope of Work

Water Technology has been commissioned by Onkaparinga Council to undertake an assessment of the local wave climate and determine design wave loads for the proposed boardwalk along Witton Bluff, Port Noarlunga.

This assessment has included:

- A collation & review of relevant meteorological & oceanographic data (wind and tide).
- An assessment of the local wave climate (which is comprised of ocean swell and locally wind generated waves) using desktop calculation methods to identify critical design load components.
- Horizontal and vertical loads critical to the proposed structure.

The report is structured as

- Chapter 2: review the existing coastal engineering documentation applicable to the site and, review project site environmental conditions (e.g., met ocean conditions) and compile a set of design parameters e.g., design life and design event.
- Chapter 3: A reference design specifies
  - Design loads on Piles
  - Considerations about boardwalk material
- Chapter 4: Safe design register outlines key considerations for detailed design

Methods and results are described herein.

2 BASIS OF DESIGN

In order to undertake this assessment, a range of data items were sourced and applied to this investigation. They are described in further detail below.

2.1 References, Manuals and Standards

The following documents are reviewed to develop this reference design report:

- WBBT Concept Design_v2 pdf.pdf
- Witton Bluff Base Trail Environmental Feasibility and Design Concept Study
- WittonNrth_Seawall_Upgrade_Design_Report_11-0669saa-pobrp-Rev A.pdf

This assessment is undertaken in accordance with the following reference documents:

- AS 4997: Guidelines for the Design of Maritime Structures:
- BS 6349 Part 1: Maritime Structures Code of Practice for General Criteria
- The Coastal Engineering Manual (USACE, 2002)
- GODA 2000 – Random Seas and Design of Marine Structure
- (J.D.Fenton, 1990) - Nonlinear Wave Theories (Fourier approximation of nonlinear wave)
- JAN WIENKE (2004) - Theoretical Formulae for Wave Slamming Loads on Slender Circular Cylinders and Application for Support Structures of Wind Turbines
- South Australian Government, 2020 Tidal Tables for South Australian Ports
- Australian National Tide Tables 2020 (ANTT)
- Coast Protection Board Policy Document Revised 29 July 2016 (South Australia)

2.2 Datum and Convention

The vertical datum is referred to Australian Height Datum (AHD), which is 1.4 m above Lowest Astronomical Tide (LAT). MSL is 1.2 m above LAT. The horizontal project in this report are referenced to GDA 1994 MGA Zone 54.

Calculations shall be in S.I. units. Loads are given in kN, kN/m or kPa (kN/m^2).

2.3 Design Life

The design life of a structure depends on its intended use, durability of the material used and maintenance requirements. Australian Standard AS4997-2005 suggests a 50-year design life for 'Normal Commercial Structures' as presented in Figure 2-1. At the end of the design life, the structure should have adequate strength to resist ultimate load and remain serviceable. A 50-year design life is adopted for the current board walk design.

DESIGN LIFE OF STRUCTURES

Facility category	Type of facility	Design life (years)
1	Temporary works	5 or less
2	Small craft facility	25
3	Normal commercial structure	50
4	Special structure/residential	100

FIGURE 2-1 DESIGN LIFE FOR STRUCUTRES AS PER AS4997-2005

2.4 Environmental Conditions

2.4.1 Tidal Plane

To produce a baseline condition for tidal and wave exacerbated loading conditions, tidal heights in relation to the Australian Height Datum (AHD) are required for investigation.

Tidal planes in the study area are obtained from the South Australian Government's Tidal Tables (2020) and Australia National Tide Table (ANTT) as shown in Table 2-1.

TABLE 2-1 PRESENT DAY TIDAL LEVELS AT PORT NOARLUNGA

Tidal Planes	SA Tidal Tables 2020 (m AHD)	ANTT (m AHD)
Highest Astronomical Tide (HAT)		+1.07
Mean High Water Springs (MHWS)	+0.6	+0.59
Mean Sea Level	-0.2	-0.2
Lowest Astronomical Tide (LAT)	-1.4	-1.33

2.4.2 Storm Tide

Previous studies performed within the Gulft St Vincent area (inclusive of Port Noarlunga) have been extrapolated to provide current and future storm tide data based on a previous reported provided by Coastal Engineering Solutions (CES, 2011). Values derived from CES 2011 are presented in Table 2-2.

TABLE 2-2 STORM TIDE TABLE

ARI	Storm tide level (m AHD)
10	2.13
25	2.24
50	2.30
100	2.35
200	2.45*

* extrapolated

2.4.3 Sea Level Rise

The Coast Protection Board adopted a 0.3m sea level rise by 2050 and 1m by 2100 for planning purposes in South Australia (Coast Protection Board, 2016). For the 50-year design life of the boardwalk, 0.6 m sea level rise allowance is adopted to cater for the water level increase from climate change up to 2070. Those sea level rise values are higher than values presented in AS 4997 Table 4.1 due to the updated sea level rise projections by the Intergovernmental Panel on Climate Change (IPCC AR5 Synthesis Report).

2.5 Design Condition

2.5.1 Design Storm Event

The design event is selected based on percentage chance of storms occurring within the design life of structures (see Table 2-3). For 50 years design life, the exposure is about 40% for a one in 100 years storm and about 22% for a one in 200 year storm. Refer to AS4997, for normal marine structures, a one in 200 years storm should be selected for design purpose, which corresponds to 22% occurrence within the design life of the proposed structure.

TABLE 2-3 PERCENTAGE CHANCE OF DESIGN STORMS OCCURING DURING THE DESIGN LIFE OF STRUCTURES

Number of years within the period	Average Recurrence Interval *(years)*					
	5	10	25	50	100	200
1	*18.1%*	*9.5%*	*3.9%*	*2.0%*	*1.0%*	*0.5%*
5	*63.2%*	*39.3%*	*18.1%*	*9.5%*	*4.9%*	*2.5%*
10	*86.5%*	*63.2%*	*33.0%*	*18.1%*	*9.5%*	*4.9%*
25	*99.3%*	*91.8%*	*63.2%*	*39.3%*	*22.1%*	*11.7%*
50	*100.0%*	*99.3%*	*86.5%*	*63.2%*	*39.3%*	*22.1%*
100	*100.0%*	*100.0%*	*98.2%*	*86.5%*	*63.2%*	*39.3%*
200	*100.0%*	*100.0%*	*100.0%*	*98.2%*	*86.5%*	*63.2%*

TABLE 2-4 ANNUAL PROBABILITY OF EXCEEDANCE OF DESIGN WAVE EVENTS (AS4997-2005)

ANNUAL PROBABILITY OF EXCEEDANCE OF DESIGN WAVE EVENTS

Function category	Category description	Design working life (years)			
		5 or less (temporary works)	25 (small craft facilities)	50 (normal maritime structures)	100 or more (special structures/ residential developments)
1	Structures presenting a low degree of hazard to life or property	1/20	1/50	1/200	1/500
2	Normal structures	1/50	1/200	1/500	1/1000
3	High property value or high risk to people	1/100	1/500	1/1000	1/2000

NOTE: The design water levels used in combination with waves determined from Table 5.4 should be taken as not below mean high water springs.

For coastal structures located in nearshore regions such as the boardwalk, wave conditions are often depth limited which leads to high level correlation between extreme water level and extreme wave height. In this study it is recommended to use 0.5% wave height AEP (or one in 200 years storm wave) in combination with a 0.5% AEP (or one in 200 years storm tide) water level. The assumption of complete dependence between waves and wave levels especially for high ARI event could be conservative while still reasonably applicable for this reference design.

2.5.2 Design Water Level

Design extreme water levels are determined based on the ARI storm tide and sea level rise scenarios (+0 m for present and +0.6 m in 2070). The design water levels are summarised in Table 2-5.

TABLE 2-5 DESIGN WATER LEVELS

ARI (Years)	Design WL (Present Data) mAHD	Design WL (2070) mAHD
10	2.13	2.73
25	2.24	2.84
50	2.30	2.9
100	2.35	2.95
200	2.45	3.05

2.5.3 Design Wave

CES 2011 undertook numerical modelling to determine wave heights off Witton Bluff. The analysis considered swell waves entering into the Gulf St Vincent and locally generated wind waves. For the swell wave analysis only about 10 years of data was available which may not be appropriate for extrapolation of more extreme events. The wind wave analysis does consider a total of 57 years of data and is appropriate to use for extrapolation for a 100-year ARI event.

Water Technology has an existing wave model of the Gulf. This has been developed for local project work and has last been refined in 2018 as part of the Aldinga Cliff stabilisation works project. This model considers data from 1955 – 2018 and is therefore considered more appropriate to be used for extrapolations for a 100-year and 200-year ARI event. The model is not sufficiently refined at the area of interest at Witton Bluff. Therefore waves have been extracted at the -10mAHD contour and transferred to the site using GODA method with consideration of wave refraction, shoaling and breaking. Two chainages (Chainage 200 m with sea bed level of 0 m AHD, and chainage 77 with sea bed level of 2.5 m AHD) have been analysed in more detail as those are the locations where the structure is most exposed to waves.

The design wave for structures should be equivalent to $H_{1\%}$ according to AS4997. In this project, the wave load will be estimated using H_{max}, given the rather small differences between $H_{1\%}$ and H_{max} for breaking waves. The final design waves are presented below in Table 2-6.

TABLE 2-6 DESIGN WAVES

ARI	Tp (s)	Present day			2070 (0.6m sea level rise)		
		Storm Tide level (mAHD)	Hs (m)	H_{max}(m)	Storm Tide level (mAHD)	Hs (m)	H_{max}(m)
Around Chainage 200							
50	9	2.30	1.7	2.3	2.9	2.1	2.7
100	9	2.35	1.8	2.4	2.95	2.1	2.8
200	9	2.45	1.8	2.5	3.05	2.2	2.9
All other sections (cliff level above 2.5 m AHD)							
50	9	2.30	0.3	0.6	2.9	0.6	1.0
100	9	2.35	0.3	0.6	2.95	0.6	1.0
200	9	2.45	0.4	0.7	3.05	0.7	1.1

Note those design waves are estimated using GODA method. Some wave heights may not exist in such a small water depth due to wave breaking. The ratio between H_b (breaking wave height) and water depth can range from ~ 0.6 over a flat bottom to ~1.2 over a steep slope for regular waves. Values are adjusted within the wave load calculations where this is the case.

2.5.4 Wave Reflection

Waves approaching vertical surface can be reflected and form a standing wave in front of the vertical cliff with wave magnitude varying by distance from the cliff.

In this project, the design wave (Tp = 9s) length is approximately 44 m (may range between 40m and 50 m for the various return periods). The estimated antinodes (location of maximal wave amplitude) are about 22 m/44 m from the cliff wall. According to concept drawings near chainage 200, some piles will be located in the wave reflection zone. However, the water depth will still be the limiting factor that governs the design wave heights.

3 REFERENCE DESIGN

3.1 Design Load

3.1.1 Wind Load

Wind load is calculated using regional wind velocities for category A1 winds in accordance of AS1170.2 Wind Actions (see Table 3-1). Velocities for the regional area were produced for 3 second wind gusts, which provides suitable wind assessment for the proposed piles.

TABLE 3-1 WIND LOADING

Design Criteria	Wind Loading Pressure
Wind 50-year ARI Event	0.91kPA
Wind 100-year ARI Event	1.01kPA
Wind 200-year ARI Event	1.11kPA

3.1.2 Wave Crest

Wave crest level has been reviewed using Fenton wave formula. The maximal wave crest can reach about 5.5 m AHD in 2070 which is about the design deck level (+5.5 m AHD). A review of wave form shows the wave crest has a very steep peak and a short duration and is very difficult to sustain in 3 m water depth. This means that this extreme wave height is more likely to break with the majority of the waves not reaching that extreme crest level.

Wave-in-deck loading is thereby not considered.

TABLE 3-2 WAVE CREST LEVELS

ARI	Storm Tide level (mAHD)	Maximal Wave Crest (m AHD)	
		Present day	2070 (0.6m slr)
Around Chainage 200			
50	2.3	4.4	5.4
100	2.35	4.6	5.5
200	2.45	4.8	5.5

3.1.3 Wave Load

In order to obtain the governing horizontal loads applied to the board walk dynamic velocity and acceleration elements were investigated through MATLAB script. This allows simulation of critical orbital velocities (U) at the wave crest and the critical acceleration (Ax) within the mid crest of the design wave. A Fourier method set by Fenton 1990 is used to obtain the peak orbital velocities, associated acceleration and wave asymmetry. Wave asymmetry associates the steepening factor of a wave as it reaches its toppling point, therefore allowing for a dynamic height analysis of our design wave based on Fenton's (1990) method.

Wave loads on piles are estimated based on following assumptions:

- Morison equation is applied given the small pile dimension relative to wavelength (Ws/L<0.2).

- Circular pile with a drag coefficient of 1.04 (per AS4997) for preliminary assessment; drag coefficient depends on ratio of pile diameter and water depth and Reynold number, which can be estimated more accurately once pile size is determined.
- Circular pile with an inertial force coefficient of 2 (per BS 6349).
- Some H_{max} shown Table 2-6 cannot be simulated by Fenton solver due to wave breaking. Design wave height is thereby reduced to be solvable as a high order wave rather than determined by GODA empirical algorithm.
- The bed level is determined as 0 m AHD for chainage 200 (representative of the cave section) and 2.5 m AHD for chainage 77 (representative of sections above the cliff).
- A selection of circular piles has been used to estimate wave loads. Values can be interpolated for different pile sizes while extrapolation should be strictly avoided. Marine growth in accordance with chapter 3.2.4 will need to be added to the pile diameter when determining loads.
- Tidal current is assumed as negligible towards the cliff face.
- Wave load is calculated based on H_{max}.

Chainage 200

The concept design of the boardwalk within the cliff embayment around chainage 200 m (bed level about 0mAHD) is shown in Figure 3-1 and Figure 3-2.

FIGURE 3-1 CLOSE UP OF CHAINAGE 200 METERS FORESHORE ENVIRONMENT

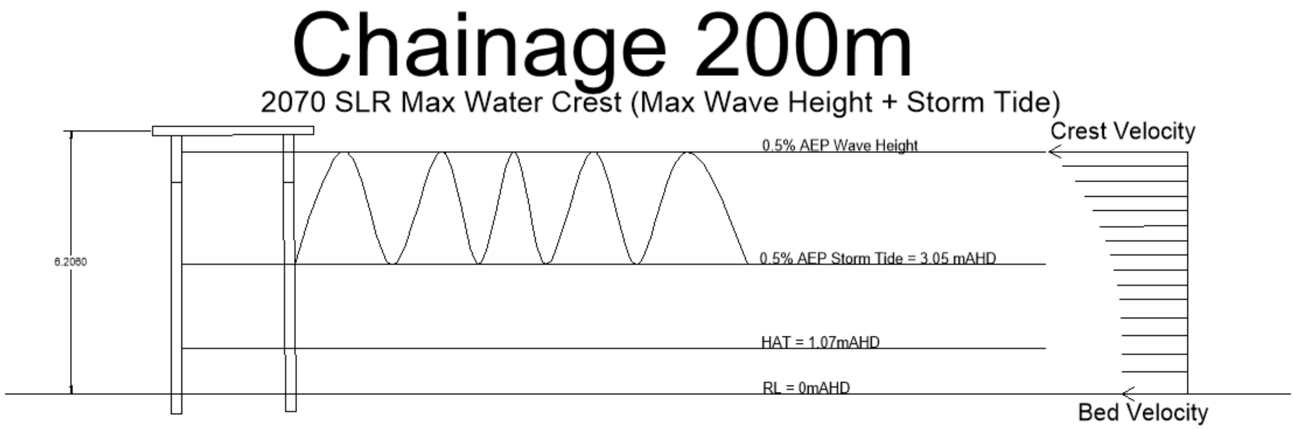

FIGURE 3-2 CHAINAGE 200MAHD MAX WATER CREST AND BOARD WALK HEIGHT

The estimated horizontal (Fx, positive toward the shore) and vertical (Fz, positive upwards) wave load at chainage 200 are presented in Table 3-3 (present day) and Table 3-4 (2070 SLR). Note the tables include only maximal values across different wave phases.

TABLE 3-3 WAVE LOAD (PER METER PILE LENGTH) A SELECTION OF PILE SIZES – PRESENT DAY, 200-YEAR ARI

z (m AHD)	Max velocity U (m/s)	Max A_x (m^2/s)	Fx (KN/m)					Fz (KN/m)				
			Ø305 mm	Ø324 mm	Ø356 mm	Ø406 mm	Ø457 mm	Ø305 mm	Ø324 mm	Ø356 mm	Ø406 mm	Ø457 mm
0.00	1.64	0.81	0.46	0.49	0.54	0.62	0.71	0.00	0.00	0.00	0.00	0.00
0.20	1.64	0.81	0.46	0.49	0.54	0.62	0.71	0.01	0.01	0.01	0.01	0.01
0.40	1.65	0.81	0.46	0.50	0.55	0.63	0.72	0.01	0.02	0.02	0.02	0.03
0.60	1.67	0.83	0.48	0.51	0.56	0.65	0.74	0.02	0.02	0.03	0.04	0.05
0.80	1.70	0.84	0.49	0.52	0.58	0.67	0.76	0.03	0.04	0.04	0.05	0.07
1.00	1.73	0.86	0.51	0.54	0.60	0.69	0.79	0.04	0.05	0.06	0.07	0.09
1.20	1.78	0.90	0.54	0.57	0.63	0.73	0.83	0.06	0.06	0.07	0.09	0.11
1.40	1.83	0.93	0.57	0.61	0.67	0.77	0.88	0.07	0.08	0.09	0.12	0.14
1.60	1.89	0.98	0.61	0.65	0.72	0.83	0.94	0.09	0.10	0.12	0.14	0.17
1.80	1.96	1.03	0.65	0.70	0.77	0.89	1.02	0.11	0.13	0.14	0.18	0.21
2.00	2.05	1.10	0.71	0.76	0.84	0.97	1.11	0.14	0.15	0.18	0.22	0.26
2.20	2.14	1.19	0.78	0.83	0.92	1.07	1.22	0.17	0.19	0.22	0.26	0.31
2.40	2.26	1.28	0.87	0.93	1.03	1.19	1.35	0.21	0.23	0.26	0.32	0.38
2.60	2.39	1.38	0.97	1.04	1.15	1.33	1.52	0.26	0.28	0.32	0.39	0.46
2.80	2.55	1.52	1.09	1.17	1.30	1.50	1.71	0.32	0.34	0.39	0.46	0.55
3.00	2.73	1.67	1.25	1.33	1.48	1.71	1.96	0.37	0.39	0.44	0.52	0.61
3.20	2.94	1.84	1.44	1.54	1.71	1.98	2.26	0.38	0.41	0.46	0.53	0.61
3.40	3.19	1.79	1.68	1.80	2.00	2.31	2.65	0.12	0.12	0.12	0.12	0.11

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3.60	3.48	1.42	1.99	2.12	2.36	2.73	3.13	-0.10	-0.12	-0.16	-0.23	-0.31
3.80	3.85	1.78	2.43	2.58	2.83	3.27	3.75	-0.04	-0.05	-0.08	-0.13	-0.20
4.00	4.29	0.00	3.02	3.21	3.53	4.03	4.53	-0.27	-0.30	-0.37	-0.48	-0.60
4.20	4.29	0.00	3.02	3.21	3.53	4.03	4.53	-0.27	-0.30	-0.37	-0.48	-0.60
4.40*	4.29	0.00	3.02	3.21	3.53	4.03	4.53	-0.27	-0.30	-0.37	-0.48	-0.60

‘

TABLE 3-4 WAVE LOAD (PER METER PILE LENGTH) FOR A SELECTION OF PILE SIZES – 2070, 200-YEAR ARI

z (m AHD)	Max U (m/s)	Max A_x (m^2/s)	Fx (KN/m)					Fz (KN/m)				
			Ø305 mm	Ø324 mm	Ø356 mm	Ø406 mm	Ø457 mm	Ø305 mm	Ø324 mm	Ø356 mm	Ø406 mm	Ø457 mm
0	1.78	0.90	0.54	0.57	0.63	0.73	0.83	0.00	0.00	0.00	0.00	0.00
0.2	1.78	0.90	0.54	0.57	0.63	0.73	0.83	0.01	0.01	0.01	0.01	0.01
0.4	1.79	0.91	0.54	0.58	0.64	0.73	0.84	0.01	0.01	0.02	0.02	0.03
0.6	1.80	0.91	0.55	0.59	0.65	0.75	0.85	0.02	0.02	0.03	0.03	0.04
0.8	1.82	0.92	0.56	0.60	0.66	0.76	0.87	0.03	0.03	0.04	0.05	0.06
1	1.85	0.94	0.58	0.61	0.68	0.78	0.89	0.04	0.04	0.05	0.06	0.08
1.2	1.88	0.96	0.60	0.64	0.70	0.81	0.92	0.05	0.05	0.06	0.08	0.10
1.4	1.91	0.99	0.62	0.66	0.73	0.84	0.96	0.06	0.07	0.08	0.10	0.12
1.6	1.96	1.02	0.65	0.69	0.76	0.88	1.00	0.08	0.08	0.10	0.12	0.15
1.8	2.01	1.06	0.68	0.73	0.80	0.92	1.05	0.09	0.10	0.12	0.14	0.17
2	2.06	1.10	0.72	0.77	0.85	0.98	1.11	0.11	0.12	0.14	0.17	0.21
2.2	2.13	1.15	0.77	0.82	0.90	1.04	1.18	0.13	0.14	0.17	0.20	0.25
2.4	2.20	1.21	0.82	0.87	0.97	1.11	1.27	0.16	0.17	0.20	0.24	0.29
2.6	2.28	1.28	0.88	0.94	1.04	1.20	1.37	0.19	0.20	0.23	0.28	0.34
2.8	2.38	1.35	0.96	1.02	1.13	1.30	1.48	0.22	0.24	0.28	0.33	0.40
3	2.48	1.45	1.04	1.11	1.23	1.42	1.62	0.26	0.28	0.32	0.39	0.46
3.2	2.60	1.55	1.14	1.22	1.35	1.56	1.77	0.31	0.33	0.38	0.45	0.54
3.4	2.74	1.67	1.26	1.35	1.49	1.72	1.96	0.37	0.40	0.45	0.53	0.63
3.6	2.89	1.79	1.40	1.50	1.66	1.91	2.18	0.42	0.45	0.51	0.60	0.69
3.8	3.06	1.93	1.57	1.67	1.85	2.14	2.44	0.45	0.48	0.53	0.62	0.71
4	3.25	2.09	1.76	1.88	2.09	2.41	2.75	0.43	0.46	0.50	0.58	0.65
4.2	3.48	2.16	2.00	2.14	2.37	2.74	3.12	0.34	0.36	0.39	0.43	0.47
4.4	3.73	2.09	2.29	2.45	2.71	3.14	3.58	0.16	0.16	0.16	0.16	0.15
4.6	4.03	1.57	2.66	2.83	3.13	3.62	4.13	-0.11	-0.14	-0.18	-0.26	-0.35
4.8	4.38	1.90	3.14	3.34	3.67	4.22	4.82	-0.05	-0.06	-0.10	-0.16	-0.23

z (m AHD)	Max U (m/s)	Max A_x (m^2/s)	Fx (KN/m)					Fz (KN/m)				
			Ø305 mm	Ø324 mm	Ø356 mm	Ø406 mm	Ø457 mm	Ø305 mm	Ø324 mm	Ø356 mm	Ø406 mm	Ø457 mm
5.0	4.38	1.90	3.14	3.34	3.67	4.22	4.82	-0.05	-0.06	-0.10	-0.16	-0.23
5.2*	4.38	1.90	3.14	3.34	3.67	4.22	4.82	-0.05	-0.06	-0.10	-0.16	-0.23

* As discussed, the extreme wave crest listed in Table 3-2 is difficult to sustain at that water depth. As such the realistic wave loading only applies to a reduced wave crest as listed in Table 3-3 and 3-4.

Chainage 77.71m

The concept design of boardwalk at chainage 77.71 m is shown in Figure 3-3 and should be used for all boardwalk sections that are founded on the cliff above 2.5 m AHD.

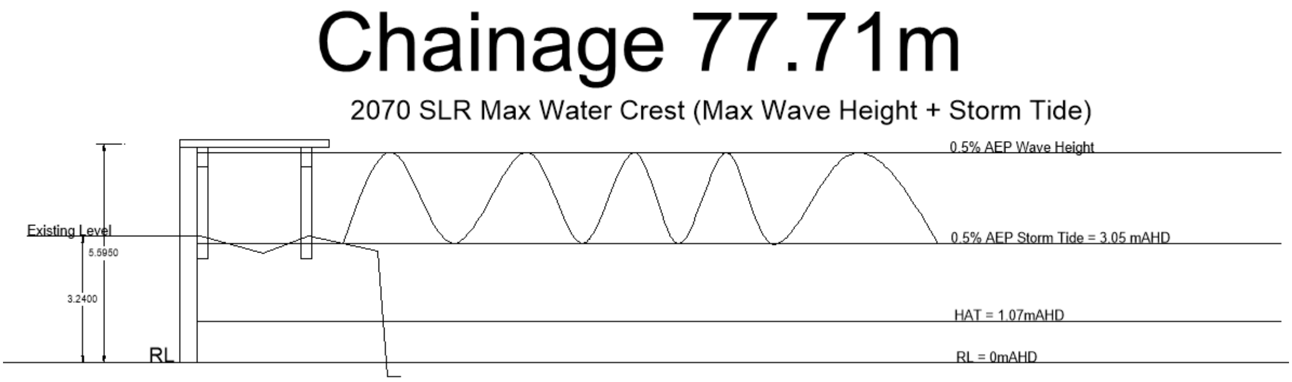

FIGURE 3-3 CHAINAGE 77.71MAHD MAX WATER CREST AND BOARD WALK HEIGHT

Wave load should be calculated that:

- If the pile is very close to the cliff offshore edge (less than one wave length (40-50m) distance), wave load presented in Table 3-3 (present day) and Table 3-4 (2070 SLR) should be used with the level of the min wave load at the level of the cliff surface. This is to avoid underestimation of wave load from direct impact of waves before breaking over the cliff.
- If the pile is located over one wavelength from the cliff, wave load should be calculated by Table 3-5. Note the table includes only maximal values across different wave phases. The present-day wave load is not presented due to compromised model accuracy from intense wave breaking (storm tide level is near the cliff level).

A review of concept drawing suggests all piles are located within one wavelength distance from the cliff. Table 3-3 (present day) and Table 3-4 (2070 SLR) should be used as an conservative approximation of wave loads on piles at all chainages.

TABLE 3-5 WAVE LOAD (PER METER PILE LENGTH) FOR A SELECTION OF PILE SIZES – 2070, 200-YEAR ARI

z (m AHD)	Max U (m/s)	Max A_x (m^2/s)	Fx (KN)					Fz (KN)				
			Ø219 mm	Ø273 mm	Ø305 mm	Ø324 mm	Ø356 mm	Ø219 mm	Ø273 mm	Ø305 mm	Ø324 mm	Ø356 mm
2.5	0.82	0.37	0.08	0.11	0.13	0.14	0.15	0.00	0.00	0.00	0.00	0.00

z (m AHD)	Max U (m/s)	Max A_x (m^2/s)	Fx (KN)					Fz (KN)				
			ø219 mm	ø273 mm	ø305 mm	ø324 mm	ø356 mm	ø219 mm	ø273 mm	ø305 mm	ø324 mm	ø356 mm
2.6	0.83	0.37	0.09	0.11	0.13	0.14	0.16	-0.01	-0.01	-0.01	-0.01	-0.02
2.7	0.87	0.39	0.09	0.12	0.14	0.15	0.17	-0.01	-0.02	-0.03	-0.03	-0.04
2.8	0.92	0.43	0.11	0.14	0.16	0.17	0.19	-0.02	-0.03	-0.04	-0.05	-0.06
2.9	1.00	0.52	0.12	0.16	0.19	0.20	0.23	-0.03	-0.05	-0.06	-0.07	-0.08
3	1.13	0.63	0.15	0.20	0.23	0.25	0.29	-0.04	-0.06	-0.08	-0.09	-0.11
3.1	1.30	0.78	0.20	0.26	0.30	0.33	0.37	-0.05	-0.08	-0.10	-0.12	-0.14
3.2	1.56	1.03	0.29	0.36	0.41	0.44	0.50	-0.07	-0.10	-0.12	-0.14	-0.17
3.3	1.93	0.00	0.44	0.55	0.61	0.65	0.71	-0.07	-0.10	-0.13	-0.14	-0.17

Chainage 464.34 m

The concept design of boardwalk at chainage 464.34 m is shown in Figure 3-4. Refer to Table 3-3 (present day) and Table 3-4 (2070 SLR) for wave loads.

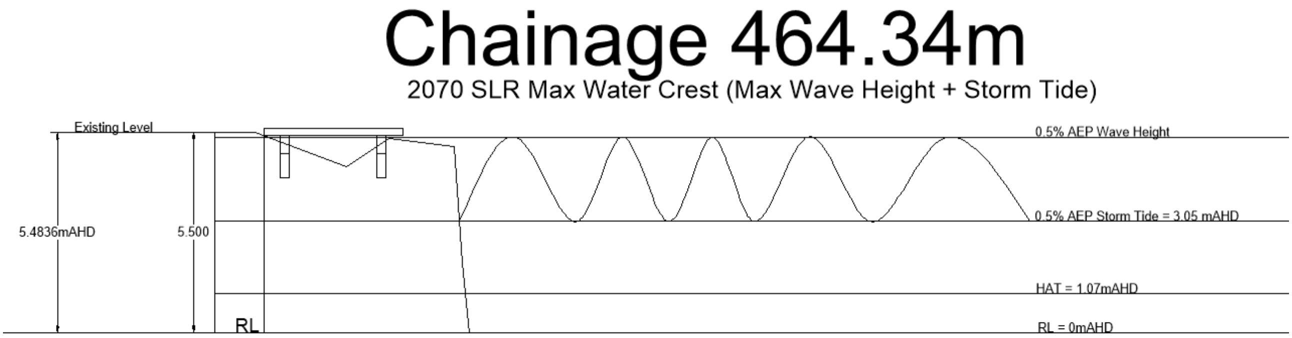

FIGURE 3-4 CHAINAGE 464.34 MAHD MAX WATER CREST AND BOARD WALK HEIGHT

Other Chainages with piles on top of the cliff (above 2.5mAHD)

Refer to Table 3-3 (present day) and Table 3-4 (2070 SLR) for wave loads.

3.1.4 Wave Slamming

The total wave forces acting on structures can be divided into the quasi-static force (Morison force, pulsating) and the slamming force (impulsive) due to breaking waves. The wave slamming forces are very large forces acting for a short period of time. The wave slamming force can be estimated according to:

- Goda (1966) formula for horizontal slamming force on cylinder which proposed a model to estimate the impact force by considering the breaking wave as a vertical wall of water hitting the cylinder at a rate of wave celerity (given as kN/m for different pile sizes considered). For pile design, this force should be evenly applied over an area between 0.6 to 1 time the wave crest height as shown in Figure 3-5. For other structure elements e.g., headstocks, a wave slamming pressure is provided in Table 3-6 which is appliable to structures located within the impact area of wave slamming as indicated by Figure 3-5.
- BS6349.1 for vertical wave slamming force. A slamming coefficient of 4.6 is used as a conservative estimate of slamming pressure in the absence of physical model test. Vertical slamming is reported as pressure (in kPa) due to the lack of details in dimension of structure elements.

Model results are presented in Table 3-6.

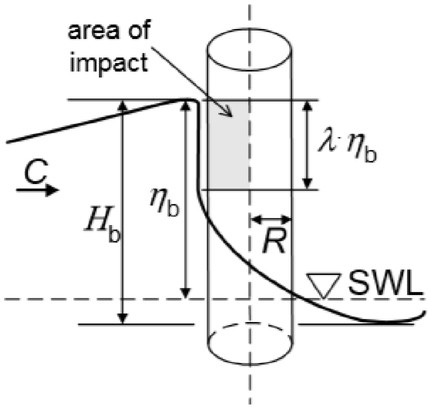

FIGURE 3-5 DIAGRAM OF HORIZONTAL WAVE SLAM (GODA 1966)

Note wave slamming load is appliable to all pile members exposed to wave crest which will not be differentiated by chainages. They are estimated based on wave condition in front of the cliff rather than the depth-limited wave on the cliff platform.

The horizontal wave slamming load should be considered as additional impulsive wave force for pile design. The vertical wave slam load can be used where horizontal structure members are to be considered at detailed design stage.

The deck is located outside of the wave zone and therefore not impacted by wave slam. However, the headstocks will be impacted. Due to the unknown dimensions of the headstocks, wave loading will need to be determined in detail during the detailed design phase. It is advisable to keep the headstock thickness limited in order to reduce wave loading onto the headstock.

TABLE 3-6 WAVE SLAM FORCES

Scenario	Wave Celerity	Max V (m/s)	Goda Horizontal Slamming force per meter pile length (KN/m)					Horizontal slam Pressure (KPa)*	Vertical slam Pressure (KPa)**
			Ø305 mm	Ø324 mm	Ø356 mm	Ø406 mm	Ø457 mm		
present, 0.5% design event	4.8	1.5	11.2	11.9	13.1	14.9	16.8	36.7	5.0
2070, 0.5% design event	5.3	1.6	14.0	14.8	16.3	18.6	20.9	45.8	6.0

*Horizontal slam force given as pressure for horizontal beams or headstocks (sizes not determined at the preparation of this report).

**Vertical slam force given as pressure for panels, horizontal beams or headstocks (sizes not determined at the preparation of this report)

3.1.5 Imposed Loads

The following imposed deck loads described in Table 3-7 will be considered.

TABLE 3-7 DECK LOAD

Parameter	Description	Source
Board Walk uniformly distributed load	5 kPa (AS 4997 Class 5 for public boardwalks), Distribution loads should be applied over the whole of the deck between kerbs.	AS 4997 Table 5.1
Board Walk concentrated load	20 kN with 1.8 m spacing (AS 4997 Class 5 for public boardwalks), Concentrated loads should be applied at critical locations in one span in lieu of a distributed load.	AS 4997 Table 5.1
Construction loads	Any construction loading beyond the limits of the loading noted above should be considered during the detailed design.	

3.1.6 Seismic Loads

Assessment will be based on a static lateral force procedure set out in AS 1700.4

3.1.7 Load Factors

Load combination factors shall be in accordance with AS 4997 and AS 1170.0. A summary of Ultimate Limit State (ULS) and Serviceability Limit State (SLS) load factors for various applied loadings for structural design are given below in Table 3-8.

TABLE 3-8 LOAD FACTORS

Load Type	Designation	ULS	SLS	Source
Dead load (destabilising)	G	1.2	1.0	AS 1170
Dead load (stabilising)	G	0.9	1.0	AS 1170
Imposed Loads	Q	1.5	1.0	AS 1170
Wave	F_{env}	1.0	1.0	AS 1170
Wind	F_{env}	1.0	1.0	AS 1170
Seismic	E_u	1.0	1.0	AS 1170

3.1.8 Combination of Actions

Load combinations will be applied to the individual actions as designated by AS1170.0 and AS4997 as noted below.

Environmental Loads (F_{env})

- $F_{env} = W_u$ ultimate wind load
- $F_{env} = F_{wave.u}$ ultimate wave load
- $F_{env} = 0.7W_u, F_{wave.u}$ ultimate wave and wind

Serviceability Limit State Combinations

- $E_{d.s} = [G, Q]$ serviceability permanent and imposed
- $E_{d.s} = [G, Q, F_{env.s}]$ serviceability permanent, imposed and environmental

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- $E_{d.s}$ = [G, $E_{s,}$ Q] serviceability permanent, earthquake and imposed

Ultimate Limit State Combinations

- $E_{d.u}$ = [1.2G, 1.5Q] ultimate permanent and imposed
- $E_{d.u}$ = [1.2G, F_{env}] ultimate permanent, maritime and environmental
- $E_{d.u}$ = [0.9G, F_{env}] ultimate permanent, maritime and environnemental
- $E_{d.u}$ = [1.2G, 0.6Q, F_{env}] ultimate permanent, imposed, maritime and environmental
- $E_{d.u}$ = [G, E_u, 0.6Q] ultimate permanent, earthquake and imposed action

3.2 Material considerations

3.2.1 Timber Piles

Timber is a natural material often being used in the design for a maximal load of 15 to 25 tons per pile. These piles can last about 30 years depending on timber quality, treatment and maintenance condition. To be used as a pile foundation it has both pros and cons:

- Reasonable cost in certain diameter and length range;
- Easy to install and uninstall;
- Small bearing capacity;
- Fragile for rough handling and hard to drive in stiff soil/sand;
- Not as durable when comparing to concrete and well-protected steel, especially in marine environment e.g., wave splashing zone;
- Subject to limitation of lengths and diameters;
- Marine growth and degradation.

For the current design at the embayment section, the pile will be located at about mean sea level. It is subject to constant wave splashing and degradation. The pile length in that section is more than 5.5 m above soil level. Timber may not be the preferred option for piles within the embayment around chainage 200 m where durability of material becomes an issue. It is however a potential option for piles located above the cliff.

Timber piles shall be designed per AS 1720-1 and AS 2159.

Timber can be a suitable design material for the deck.

3.2.2 Steel Piles

Steel piles have a large bearing capacity and can penetrate through stiff soil layers without causing too much soil displacements. It is however relatively expensive and can be damaged by corrosion and electrolysis.

Pipe piles is a good option as it offers:

- Reasonable cost and variety of options in sizes;
- Easy to install and tough material for rough handling;
- If treated properly with coating and cathodic protection, it becomes durable and can sustain wave splashing and marine growth;
- Option to use close-end pipes for better durability;

- Some maintenance is required to reach the 50 years design life.

Steel piles are a good option for the embayment section where long pile lengths are required. Steel grade is referred to manufacturers' manual. The steel structural (including piles) design favours the use of closed form sections where practical to minimize maintenance requirements due to atmospheric corrosion. 4 mm thick end caps are nominally to be welded to the end of all sections to seal off internal surfaces. All structural connections shall be fully welded unless impractical.

Steel pipe piles shall be designed per AS 4100 and AS 2159.

Steel or plastic mesh could be considered for the deck material.

3.2.3 Concrete Piles

Concrete piles can also be considered for both, the sections of the boardwalk on top of the cliff and in the embayment. Concrete can achieve the 50 year design life with appropriate maintenance, however, construction quality and precision (e.g. reinforcement cover) can be essential in achieving a longer design life. Concrete might also be considered for the headstocks.

3.2.4 Marine growth

An allowance of 100 mm for marine growth around piles and submerged members will be considered with reference Section C.8.1 of ISO 19901.

4 SAFETY IN DESIGN

A safety in design (SiD) register has been established. This SiD register will need to be updated throughout all project phases. The SiD register uses the following risk matrix as outlined in Figure 4-1.

Risk Matrix				Severity (Where an event has more than one 'Loss Type', choose the 'Consequence' with the highest rating)				
	Loss Type (Additional 'Loss Types' may exist for an event; identify & rate accordingly)			Insignificant 1	Minor 2	Moderate 3	Major 4	Catastrophic 5
Likelihood	Certain 5	The event is expected to occur in most circumstances	>90%	5	10	15	20	25
	Very Likely 4	The event will probably occur in most circumstances.	>50–90%	4	8	12	16	20
	Likely (3)	The event might occur, but not expected to occur under normal circumstances.	>10–50%	3	6	9	12	15
	Unlikely 2	The event could occur at some time, but only in unusual circumstances	>2–10%	2	4	6	8	10
	Highly Unlikely 1	The event could occur only in exceptional circumstances	<2%	1	2	3	4	5

FIGURE 4-1 RISK MATRIX

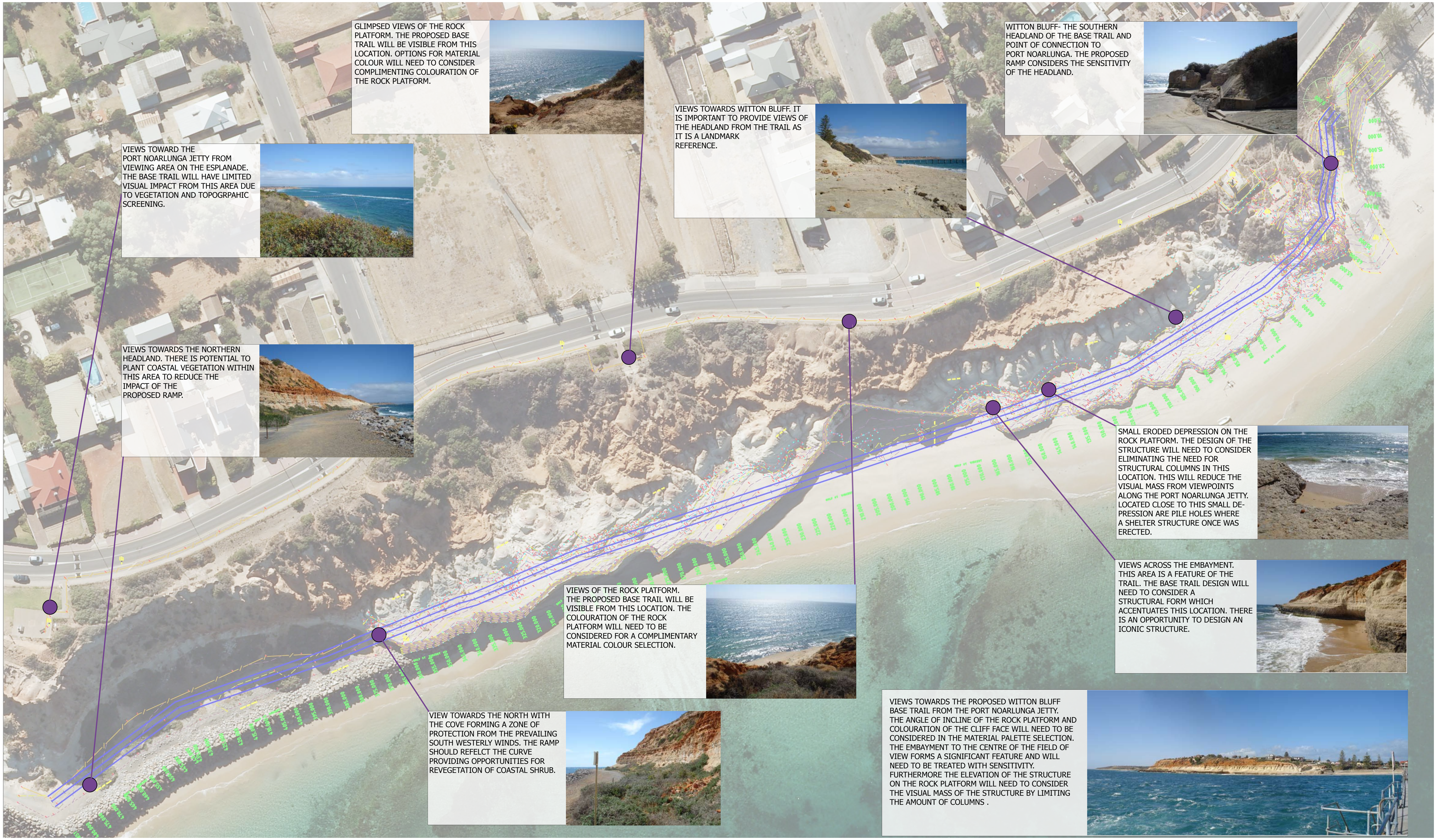
TABLE 4-1 DESIGN SAFETY REGISTGER

	Hazard Identification			Risk Assessment							Risk Control Implementation					
ID	Potential Hazard	Possible Causes Identified	Those who are affected	*Pre-control* S	L	RF	Control Measures	*Post-control* S	L	RF	Risk Owner	Implement Control - Y/N?	Action Owner(s)	Timing / Date	Status (Open / Closed)	Additional Comments
Design Phase																
1.01	Approvals - limitation and restriction on project due	Insufficient liaison with stakeholders, not conforming to relevant design requirements, policy, laws, restrictions and consenting	Onkaparinga Council	1	3	3	Adhere to all requirements	1	1	1	Onkaparinga Council					
1.02	Structural failure	Geotechnical instability	Onkaparinga Council/ Contractor	4	4	16	Regular QA, instructed by a specific QA procedure. Qualified geotechnical consultant to prepare the design and supervise construction.	4	2	8	Onkaparinga Council					
1.03	Hydraulic stability of piles	Scour developing around the piles	Onkaparinga Council/ Contractor	4	4	16	Regular QA, instructed by a specific QA procedure. Expected scour depth and anticipated erosion to be determined during the design phase and allowance to be included in the design. Onkaparinga Council to monitor scour as per the Asset Management Plan.	4	2	8	Onkaparinga Council					
1.04	Drowning due to wave overtopping, storm surge	Coastal flooding risk is significant, particularly as sea levels are rising	Onkaparinga Council/ Contractor	5	1	5	The boardwalk has to be designed to a 200-year storm event, including sea level rise. The deck will not be inundated in those design conditions. Should design conditions be exceeded structural limits may be exceeded, likely causing damage to the structure and exposing people to increased risk. Closure of the boardwalk during expected severe conditions shall be considered by Council.	4	1	4	Onkaparinga Council					
1.05	Delay to project and damage to plant due to uncertain geotechnical conditions	Insufficient fill/compaction/foundation	Onkaparinga Council/ Contractor	3	3	9	Site specific geotechnical conditions will need to be considered during the design phase.	3	2	6						
Construction Phase																
2.01	Sun/heat	UV exposure, working in heat	Contractor	4	3	12	Appropriate long PPE to be worn by all personnel. Specific CEMP and Safe Work Method Statement.	4	1	4	Contractor					
2.02	Working on/near water, drowning	Plant and personnel working in close proximity to water	Contractor	5	3	15	Works to be undertaken during appropriate sea conditions only, safe work procedures to be detailed by contractor and sub-contractors. Contractors to monitor waves and tide conditions.	5	1	5	Contractor					
2.03	Extreme weather	Prior to each working day, site manager is to check the BoM (website) and local news (internet, TV, radio etc) for extreme weather warnings	Contractor	5	3	15	Site works are to be cancelled if extreme weather is forecasted, or weather warnings are issued by the BoM.	5	1	5	Contractor					

2.04	Accidents during road transport of plant, equipment and material to/from site	Incorrect loading of haulage trucks, insufficient planning, insufficient training of operators	Onkaparinga Council/ Contractor/ Public	5	3	15	Safe Work Method Statement and CEMP and detailing of Haulage and Traffic Management Plans.	5	1	5						
2.05	Tides	Tides and wave run-up flooding plant and equipment, damaging unfinished works	Contractor	5	4	20	Work to be scheduled around tides, and site to be constructed to take into account tide levels, especially for toe maintenance.	5	1	5	Contractor					
2.06	Interaction between heavy plant, vehicles and the general public	Member of the public enters site without knowledge, or disobeys signage and barricades, causing injury or accident	Contractor/ Public	5	3	15	Traffic and pedestrian Management Plans to be compiled in the CEMP and followed by contractor and sub-contractor. Notices to be posted in local media detailing the works undertaken and the duration. Solid barricading of the work exclusion zone, flashing lights on all mobile equipment. Appropriate warning signs on barricade detailing the works undertaken and hazards present, and includes emergency phone number.	5	1	5	Contractor					
2.07	Vehicle accident during haulage of materials to site	Poor route planning, inexperienced operators, unsuited haulage vehicles for task	Contractor, Public	5	2	10	Safe Work Method Statement and CEMP and detailing of Haulage and Traffic Management Plans. Appropriate trucks and plant for task.	5	1	5	Contractor					
2.08	Interaction between heavy plant, vehicles, personnel and equipment	Poor site planning, lack of appropriate communication equipment, poor visibility	Contractor	5	3	15	Exclusion zone around heavy plant in operation. Spotters posted where required. Appropriate SWMS and regular site safety briefings. Appropriate communications systems. Temporary fences.	5	1	5	Contractor					
2.09	Interaction between heavy plant, construction activities and other water users	Insufficient public information provided, poor signage	Onkaparinga Council/ Contractor/ Public	5	2	10	Exclusion zone set around site works. Information on the works to be disseminated to the public using local media sources and local marine organisations.	5	1	5						
2.10	Changes to site following tides	Movement of sand following tides or cliff erosion, resulting in changed site conditions from day-to-day	Contractor	5	3	15	Site inspection to be conducted prior to commencing work each day, any new hazards to be addressed at prestart meeting. New controls put in place where required.	5	1	5	Contractor					
2.11	Public/ Contractor injured by cliff movement	Placement of stockpiling and machinery	Onkaparinga Council/ Contractor/ Public	5	3	15	Traffic and Pedestrian Management Plans to be compiled in the CEMP and followed by contractor and sub-contractor. Notices to be posted in local media detailing the works undertaken and the duration. Solid barricading of the work exclusion zone, flashing lights on all mobile equipment. Appropriate warning signs on barricade detailing the works undertaken and hazards present, and includes emergency phone number. Appropriate site planning for stockpile areas and machinery.	5	2	10	Contractor					
Operations Phase																
3.01	Public injury due to slips, trips and falls	Public access to structure	Public	5	3	15	Board walk structural support members and deck to be inspected regularly to ensure adequate condition for public access. This should be documented by structural and geotechnical engineer sign off.	5	1	5	Onkaparinga Council					

3.02	Structure failure	Geotechnical instability	Onkaparinga Council/ Contractor	**5**	**4**	**20**	Regular QA, instructed by a specific QA procedure	**5**	**2**	**10**	Onkaparinga Council					
3.03	Hydraulic stability	Scour/erosion	Onkaparinga Council/ Contractor	**5**	**4**	**20**	Regular QA, instructed by a specific QA procedure. Asset Management Plan.	**5**	**2**	**10**	Onkaparinga Council					
3.04	Drowning due to wave overtopping, storm surge	Lack of forecast monitoring, not closing structure during severe weather	Onkaparinga Council	**5**	**2**	**10**	The boardwalk is designed to a 200yr storm event including sea level rise. Closure of the boardwalk prior to severe storm conditions shall be implemented.	**5**	**1**	**5**	Onkaparinga Council					
Maintenance Phase																
4.01	Degradation of materials	insufficient maintenance, insufficient construction detail	Onkaparinga Council	**5**	**4**	**20**	Periodic inspections, and inspections after extreme conditions to trigger maintenance works as required	**5**	**2**	**10**	Onkaparinga Council					
4.02	Vandalism	Public access	Onkaparinga Council	**4**	**3**	**12**	Periodic inspections	**4**	**2**	**8**	Onkaparinga Council					
4.03	Cliff instabilities	Severe storm events	Onkaparinga Council	**5**	**4**	**20**	Periodic inspections, and inspections after extreme conditions to trigger maintenance works as required	**5**	**3**	**15**	Onkaparinga Council					
Demolition Phase																
5.01	Hazards, causes and risks as per construction, operation and maintenance phases.						Control measures as per construction, operation and maintenance phases.									

APPENDIX A
CONCEPT DRAWINGS



City of Onkaparinga

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NOT TO SCALE

N

WITTON BLUFF BASE TRAIL
SITE ANALYSIS

10.10.08

08146SK01B

 244 GILBERT ST ADELAIDE SA 5000 TEL (08) 8212 2679 FAX (08) 8212 3162 mail@swanburypenglase.com www.swanburypenglase.com

LEGEND

PROPOSED PATH ALIGNMENT

POTENTIAL AREAS FOR BREAKOUT SPACE INCORPORATING SEATING, ART WORKS AND SIGNAGE

VISUALLY SENSITIVE AREAS

LIMIT THE NUMBER OF COLUMNS WITHIN THESE ZONES TO ACCENTUATE THE EMBAYMENT AND SMALL DEPRESSION IN THE ROCK PLATFORM. THIS WILL ACCENTUATE THE NOTION OF FLIGHT AND THE STRUCTURE AS TREADING LIGHTLY ON THE LANDSCAPE.

POSSIBLE AREAS OF COASTAL VEGETATION

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WITTON BLUFF BASE TRAIL
LANDSCAPE CONCEPT

10.10.08

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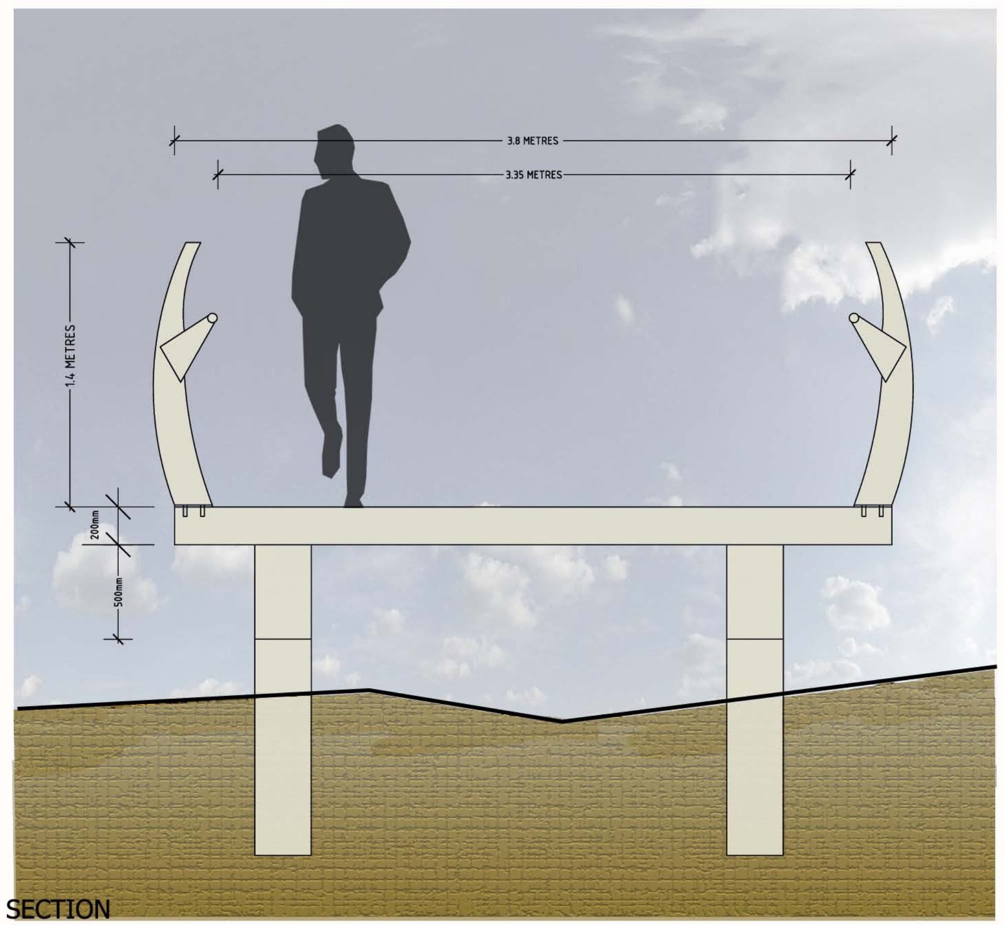
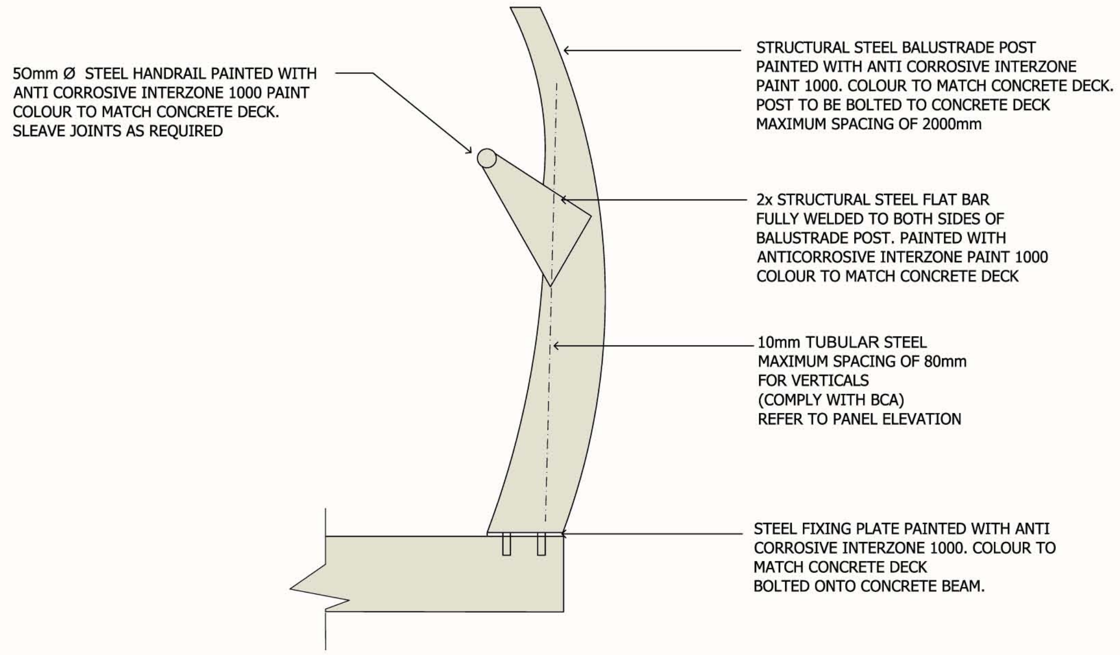
ELEVATION
CONCRETE DECK
SCALE 1:20@A1



SECTION
CONCRETE DECK
SCALE 1:20@A1



SECTION
BALUSTRADE DETAIL
SCALE 1:10@A1

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WITTON BLUFF BASE TRAIL
CONCRETE DECK

10.10.08

08146 SK05C

PHOTOMONTAGE- ARTISTS IMPRESSION NOT TO SCALE

City of Onkaparinga

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WITTON BLUFF BASE TRAIL
PHOTOMONTAGE

11.11.08

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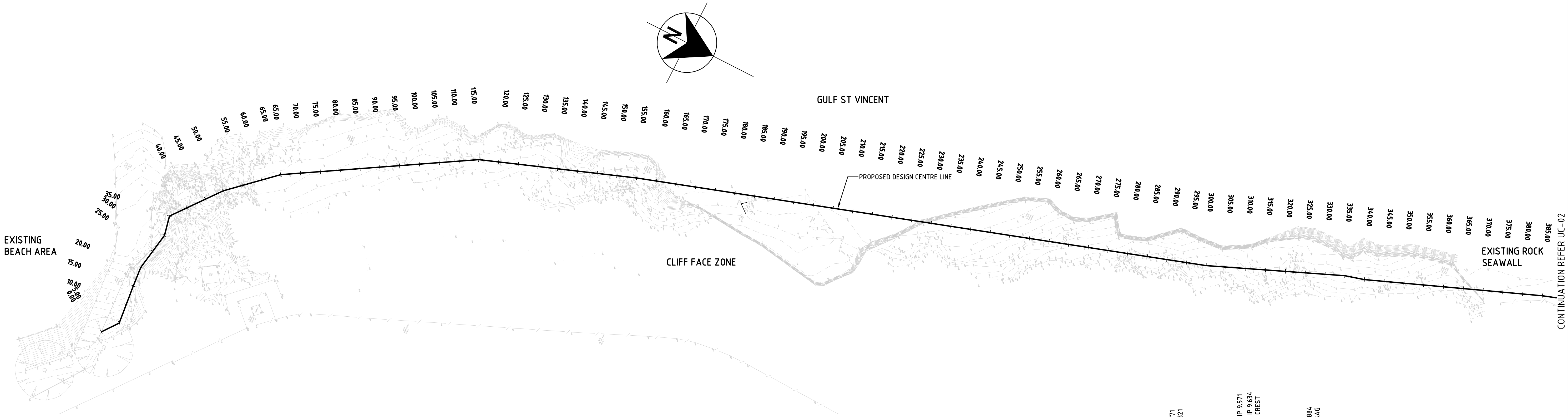
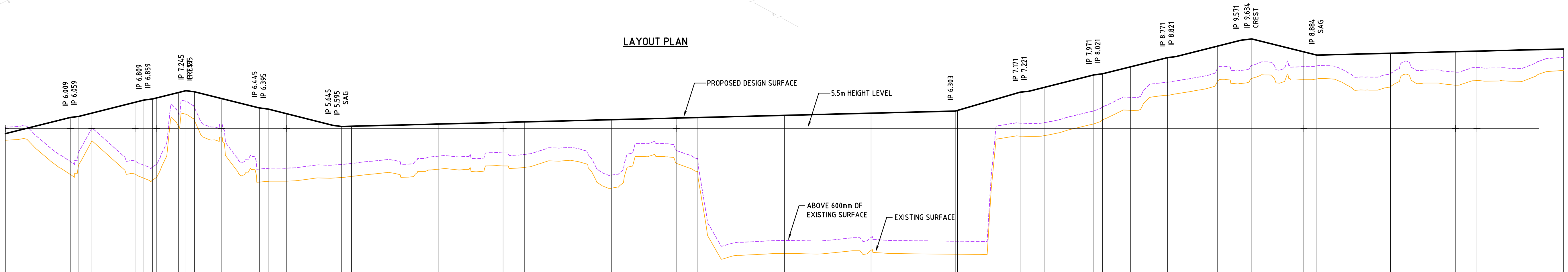


LAYOUT PLAN



HORIZONTAL CURVE

VERTICAL CURVE

GRADE %: 5% | 2.5% | 5% | 2.5% | 5% | -2.5% | -5% | -2.5% | -5% | -2.5% | 0.5% | 5.787% | 2.5% | 5% | 2.5% | 5% | 2.5% | 5% | 2.5% | -5% | 0.5%

Datum RL. -6.00

DESIGN CENTRELINE LEVEL	5.259	5.509	6.009	6.009	6.059	6.209	6.709	6.809	6.859	6.909	7.159	7.245
EXISTING SURFACE PLUS 600	5.559	5.597	3.981	3.979	4.436	5.522	4.002	3.808	3.722	3.848	6.185	6.756
NATURAL SURFACE LEVEL	4.959	4.997	3.381	3.379	3.836	4.922	3.402	3.208	3.122	3.248	5.585	6.156
CUT/FILL DEPTH	-0.300	-0.512	-2.629	-2.630	-2.223	-1.287	-3.307	-3.601	-3.737	-3.661	-1.574	-1.089
CENTRELINE CHAINAGE	0.00	5.00	15.00	15.01	17.00	20.00	30.00	32.00	34.00	35.00	40.00	41.71

DESIGN CENTRELINE LEVEL	7.195	6.880	6.445	6.412	6.395	6.180	5.645	5.595	5.606	5.706	5.781	5.806
EXISTING SURFACE PLUS 600	6.461	5.700	3.628	3.649	3.661	3.664	3.810	3.840	3.889	4.222	4.368	4.307
NATURAL SURFACE LEVEL	5.861	5.100	3.028	3.049	3.061	3.064	3.210	3.240	3.289	3.622	3.768	3.707
CUT/FILL DEPTH	-1.334	-1.780	-3.417	-3.363	-3.333	-3.116	-2.435	-2.354	-2.317	-2.084	-2.013	-2.098
CENTRELINE CHAINAGE	43.71	50.00	58.71	60.00	60.71	65.00	75.71	77.71	80.00	100.00	115.00	120.00

DESIGN CENTRELINE LEVEL	5.906	5.981	6.006	6.106	6.206	6.303	6.333	7.171	7.221	7.397	7.971	8.021
EXISTING SURFACE PLUS 600	3.346	4.495	4.053	0.330	0.472	0.300	0.299	5.745	5.738	5.765	6.313	6.495
NATURAL SURFACE LEVEL	2.746	3.895	3.453	-0.270	-0.128	-0.300	-0.301	5.145	5.138	5.165	5.713	5.895
CUT/FILL DEPTH	-3.160	-2.085	-2.553	-6.376	-6.334	-6.603	-6.634	-2.026	-2.083	-2.232	-2.258	-2.126
CENTRELINE CHAINAGE	140.00	155.00	160.00	180.00	200.00	219.47	220.00	234.47	236.47	240.00	251.47	253.47

DESIGN CENTRELINE LEVEL	8.347	8.771	8.821	9.297	9.571	9.634	9.033	8.884	8.969	9.044	9.069	9.169
EXISTING SURFACE PLUS 600	6.938	7.634	7.697	8.285	8.176	8.410	8.354	8.388	8.016	8.100	8.311	8.965
NATURAL SURFACE LEVEL	6.338	7.034	7.097	7.685	7.576	7.810	7.754	7.788	7.416	7.500	7.711	8.365
CUT/FILL DEPTH	-2.009	-1.737	-1.724	-1.612	-1.995	-1.824	-1.279	-1.096	-1.553	-1.544	-1.358	-0.804
CENTRELINE CHAINAGE	260.00	268.47	270.47	280.00	285.47	287.98	300.00	302.98	320.00	335.00	340.00	360.00

LONGITUDINAL SECTION OP 6 DES BWALK STR CL

H. 1:500
V. 1:100

PRELIMINARY - FOR REVIEW
NOT FOR CONSTRUCTION

Rev.	Date	Revision Details	Drn	Ver.	App.
01	28.10.08	FOR REVIEW	SY		

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Client: CITY OF ONKAPARINGA

Project: WITTON BLUFF PORT NOARLUNGA

	Signed	Date
Drawn SY		28.10.08
Designed SY		28.10.08
Verified		
Approved		

Drawing Title: LAYOUT PLAN AND LONGITUDINAL SECTION - 1

Project No. 35149.001

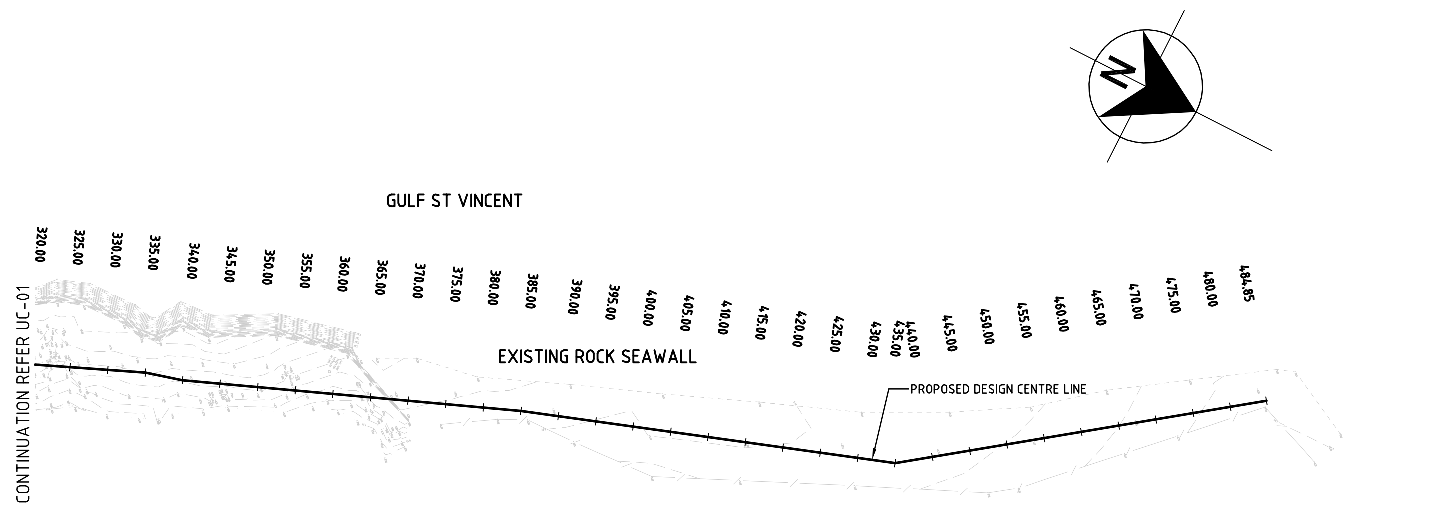
Scale 1:100 V 1:500 H

Sheet Size A1

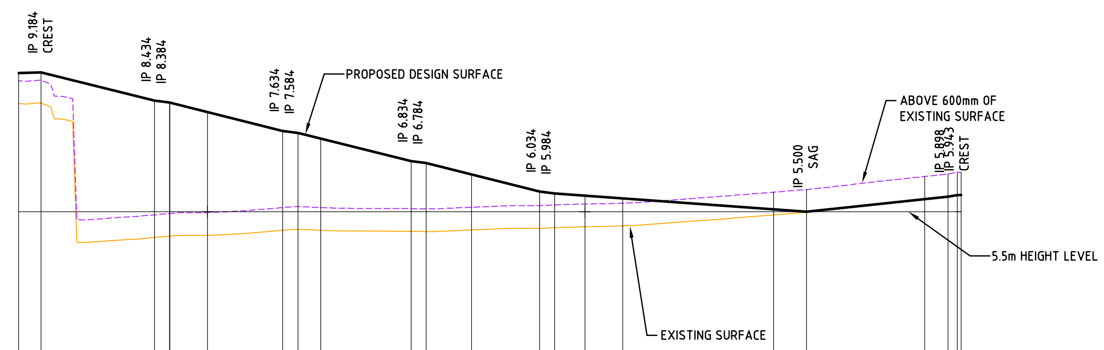
Drawing No. UC-01

Rev. 01

PLOT DATE=11/7/2008 11:28 AM SCALE=1:1 USER=SOPHIA YAN FILENAME=W:\35149-001\UC 01.DWG



LAYOU PLAN



HORIZONTAL CURVE

VERTICAL CURVE

GRADE %: 0.5% | -5% | -2.5% | -5% | -2.5% | -5% | -2.5% | -5% | -2.5% | -1.45% | 2.121% | 3.697% | 2%

Datum RL. 0.00

DESIGN CENTRELINE LEVEL	9.169	9.184	8.434	8.384	8.383	8.133	7.634	7.584	7.433	6.834	6.784
EXISTING SURFACE PLUS 600	8.965	8.969	5.418	5.452	5.453	5.475	5.609	5.635	5.611	5.580	5.577
NATURAL SURFACE LEVEL	8.365	8.369	4.818	4.852	4.853	4.875	5.009	5.035	5.011	4.980	4.977
CUT/FILL DEPTH	-0.804	-0.815	-3.615	-3.531	-3.530	-3.258	-2.624	-2.548	-2.422	-1.854	-1.806
CENTRELINE CHAINAGE	360.00	362.98	377.98	379.98	380.00	385.00	394.98	396.98	400.00	411.98	413.98

DESIGN CENTRELINE LEVEL	6.483	6.034	5.984	5.925	5.853	5.563	5.500	5.832	5.898	5.943	5.938
EXISTING SURFACE PLUS 600	5.617	5.662	5.676	5.704	5.725	6.016	6.086	6.432	6.498	6.543	6.538
NATURAL SURFACE LEVEL	5.017	5.062	5.076	5.104	5.125	5.416	5.486	5.832	5.898	5.943	5.938
CUT/FILL DEPTH	-1.466	-0.971	-0.907	-0.821	-0.728	-0.147	-0.014	0.000	0.000	-0.000	0.000
CENTRELINE CHAINAGE	420.00	428.98	430.98	435.00	440.00	460.00	464.34	480.00	483.09	484.34	484.85

LONGITUDINAL SECTION OP 6 DES BWALK STR CL

H. 1:500
V. 1:100

PRELIMINARY - FOR REVIEW
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Rev.	Date	Revision Details	Drn	Ver.	App.
01	28.10.08	FOR REVIEW	SY		

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Client: CITY OF ONKAPARINGA

Project: WITTON BLUFF PORT NOARLUNGA

Drawn	Signed	Date
SY		28.10.08
Designed SY	Signed	Date 28.10.08
Verified	Signed	Date
Approved	Signed	Date

Drawing Title: LAYOUT PLAN AND LONGITUDINAL SECTION - 2

Project No. 35149.001

Scale 1:100 V 1:500 H

Sheet Size A1

Drawing No. UC-02

Rev. 01

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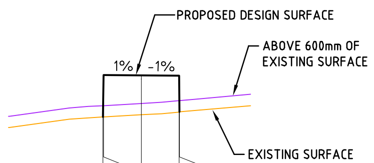
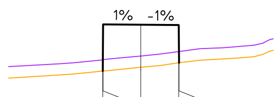


Centreline Data
X = 4719.962
Y = 2974.113
Z = 5.706
Datum 1

DESIGN HEIGHT	3.505	5.726	5.706	5.686	3.771
EXISTING SURFACE PLUS 600	4.105	4.106	4.222	4.370	4.371
DEPTH	0.000	-2.220	-2.084	-1.916	-0.000
EXISTING SURFACE	3.505	3.506	3.622	3.770	3.771
DESIGN OFFSET	-2.022	-2.000	0.000	2.000	2.019

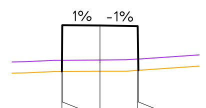
CHAINAGE 100.000



Centreline Data
X = 4668.435
Y = 3114.628
Z = 7.897
Datum 4

DESIGN HEIGHT	5.441	7.917	7.897	7.877	5.876
EXISTING SURFACE PLUS 600	6.041	6.044	6.242	6.474	6.476
DEPTH	-0.000	-2.474	-2.256	-2.004	-0.000
EXISTING SURFACE	5.441	5.444	5.642	5.874	5.876
DESIGN OFFSET	-2.025	-2.000	0.000	2.000	2.020

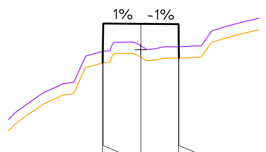
CHAINAGE 250.000



Centreline Data
X = 4616.208
Y = 3255.134
Z = 7.433
Datum 3

DESIGN HEIGHT	4.996	7.453	7.433	7.413	5.065
EXISTING SURFACE PLUS 600	5.596	5.596	5.611	5.663	5.665
DEPTH	0.000	-2.456	-2.422	-2.349	0.000
EXISTING SURFACE	4.996	4.996	5.011	5.063	5.065
DESIGN OFFSET	-2.025	-2.000	0.000	2.000	2.023

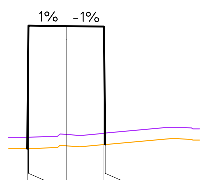
CHAINAGE 400.000



Centreline Data
X = 4748.314
Y = 2933.177
Z = 6.88
Datum 0

DESIGN HEIGHT	4.806	6.900	6.880	6.860	5.065
EXISTING SURFACE PLUS 600	5.406	5.410	5.700	5.665	5.665
DEPTH	0.000	-2.090	-1.780	-1.795	-0.000
EXISTING SURFACE	4.806	4.810	5.100	5.065	5.065
DESIGN OFFSET	-2.021	-2.000	0.000	2.000	2.018

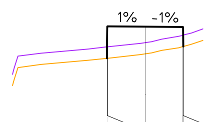
CHAINAGE 50.000



Centreline Data
X = 4684.115
Y = 3067.151
Z = 6.206
Datum -2

DESIGN HEIGHT	-0.285	6.226	6.206	6.186	-0.064
EXISTING SURFACE PLUS 600	0.315	0.316	0.472	0.530	0.536
DEPTH	-0.000	-6.510	-6.334	-6.256	0.000
EXISTING SURFACE	-0.285	-0.284	-0.128	-0.070	-0.064
DESIGN OFFSET	-2.065	-2.000	0.000	2.000	2.063

CHAINAGE 200.000

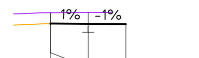


Centreline Data
X = 4634.111
Y = 3208.462
Z = 9.119
Datum 4

DESIGN HEIGHT	7.442	9.139	9.119	9.099	8.067
EXISTING SURFACE PLUS 600	8.042	8.044	8.271	8.664	8.667
DEPTH	-0.000	-1.695	-1.448	-1.035	-0.000
EXISTING SURFACE	7.442	7.444	7.671	8.064	8.067
DESIGN OFFSET	-2.017	-2.000	0.000	2.000	2.010

CHAINAGE 350.000

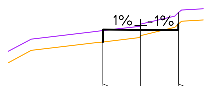
Centreline Data
X = 4575.109
Y = 3328.269
Z = 5.938



Datum 4

DESIGN HEIGHT	5.931	5.958	5.938	
EXISTING SURFACE PLUS 600	6.531	6.531	6.538	5.918
DEPTH	-0.000	-0.027	0.000	
EXISTING SURFACE	5.931	5.931	5.938	
DESIGN OFFSET	-2.000	-2.000	0.000	2.000

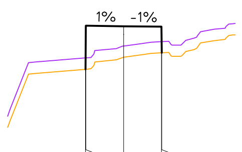
CHAINAGE 484.851



Centreline Data
X = 4793.845
Y = 2921.909
Z = 5.259
Datum 2

DESIGN HEIGHT	4.620	5.279	5.259	5.239	5.566
EXISTING SURFACE PLUS 600	5.220	5.221	5.559	6.163	6.166
DEPTH	-0.000	-0.658	-0.300	0.324	-0.000
EXISTING SURFACE	4.620	4.621	4.959	5.563	5.566
DESIGN OFFSET	-2.007	-2.000	0.000	2.000	2.003

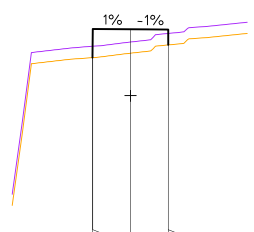
CHAINAGE 0.000



Centreline Data
X = 4699.97
Y = 3019.734
Z = 5.956
Datum -1

DESIGN HEIGHT	3.667	5.976	5.956	5.936	4.537
EXISTING SURFACE PLUS 600	4.267	4.269	4.904	5.136	5.137
DEPTH	0.000	-2.306	-1.652	-1.400	0.000
EXISTING SURFACE	3.667	3.669	4.304	4.536	4.537
DESIGN OFFSET	-2.023	-2.000	0.000	2.000	2.014

CHAINAGE 150.000

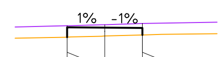


Centreline Data
X = 4652.754
Y = 3162.106
Z = 9.033
Datum -2

DESIGN HEIGHT	7.526	9.053	9.033	9.013	8.204
EXISTING SURFACE PLUS 600	8.126	8.127	8.354	8.803	8.804
DEPTH	-0.000	-1.526	-1.279	-0.809	-0.000
EXISTING SURFACE	7.526	7.527	7.754	8.203	8.204
DESIGN OFFSET	-2.015	-2.000	0.000	2.000	2.008

CHAINAGE 300.000

Centreline Data
X = 4595.862
Y = 3300.271
Z = 5.708



Datum 4

DESIGN HEIGHT	5.203	5.728	5.708	5.688	5.316
EXISTING SURFACE PLUS 600	5.803	5.803	5.859	5.916	5.916
DEPTH	-0.000	-0.525	-0.449	-0.372	-0.000
EXISTING SURFACE	5.203	5.203	5.259	5.316	5.316
DESIGN OFFSET	-2.005	-2.000	0.000	2.000	2.004

CHAINAGE 450.000

PRELIMINARY - FOR REVIEW
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Rev.	Date	Revision Details	Drn	Ver.	App.
01	28.10.08	FOR REVIEW	SY		

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Client: CITY OF ONKAPARINGA

Project: WITTON BLUFF
PORT NOARLUNGA

Drawn SY	Signed	Date 28.10.08
Designed SY	Signed	Date 28.10.08
Verified	Signed	Date
Approved	Signed	Date

Drawing Title: CROSS SECTIONS

Project No. 35149.001
Scale 1:200 V 1:200 H
Sheet Size A1
Drawing No. UC-03
Rev. 01

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STYLE= ----

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