Detailed Design Report (Stage 3)

Quinns Beach Long Term Coastal Management Study

59915802

Prepared for
City of Wanneroo

6 July 2017

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Contact Information

Cardno (WA) Pty Ltd
Trading as Cardno
ABN 77 009 119 000

11 Harvest Terrace, West Perth WA 6005

Telephone: 08 9273 3888
Facsimile: 08 9486 8664
International: +61 8 9273 3888

wa@cardno.com.au
www.cardno.com

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Executive Summary

The Quinns Beach Long Term Coastal Management Study was established by the City of Wanneroo with the goal to assess potential coastal management options to mitigate the ongoing trend of erosion present at Quinns Beach. Erosion and coastal management has a long history at Quinns Beach; early anecdotal evidence suggests that Quinns Beach has been experiencing erosion since at least the 1940s.

This report summarises Stage 3 of the Quinns Beach Long Term Coastal Management Study. The purpose of Stage 3 of the study is to prepare a detailed design based on the option selected by the City at the end of Stage 2 (Cardno 2016). After substantial community and stakeholder engagement undertaken as part of Stage 2 of the study the City selected the option referred to as MCA2 (LITPACK Option 12) in the Stage 2 report as its preferred option to for detailed design. This option was developed in Stage 2 to include:

> Extension of Groyne 2 to 120m length,

> Extension of Groyne 3 to 75m length, and

> Construction of new Groyne 4 to 60m length.

Stage 3 involved refining the chosen layout by utilising the calibrated numerical wave (SWAN) and longshore sediment transport and shoreline models (LITPACK), but extending the simulation period to 50 years for a range of groyne refinements (length and location). The performance of the refined layout under short term storm erosion (a combination of cross shore and longshore transport) was also tested for three storms that occurred in September 2013 using XBEACH.

Based on the results of the numerical modelling undertaken, a refined layout was selected which optimised the performance of the groynes across all sections of Quinns Beach whilst prioritising the sections of the beach currently impacted by ongoing erosion issues (Sections 3 and 5). The optimum refined layout was found to be very similar to the layout as it was at the end of Stage 2, and consists of the following:

> Extension of Groyne 2 to 105m length,

> Extension of Groyne 3 to 75m length, and

> Construction of new Groyne 4 to 60m length.

A detailed design of each groyne in this refined layout was prepared based on simulation of extreme wave and water level combinations. Structural design of each groyne was undertaken and the armour classes required were based on that of the existing groynes to maximise the re-use of armour material from the existing groynes. Based on the structural design calculations an additional, larger armour class of 6.6T was required for the head of each of the groynes to meet the damage criterion during the design event.

A geophysical investigation was undertaken along five sections of Quinns Beach. The purpose of the geophysical investigation was to investigate the presence (or absence) of hard substrate beneath the beach and dunes that might act as a physical barrier to potential future coastal erosion in each of the areas surveyed.

As part of this stage of the project, a detailed design of a concrete access path for the new Groyne 4 as well as concept designs of beach access structures at Groyne 1 and Waterland Point were prepared. This stage also examined the potential need for a fifth groyne and a seawall in front of the carpark at some point in the future. Recommendations on staging of the construction of the refined layout are provided as well as on ongoing monitoring requirements which will be very important for measuring the performance of the refined layout.

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1 Introduction

Quinns Beach is an iconic coastal region of the City of Wanneroo and has a long history of beach-side culture; its public amenity and recreational value are a focal point for the community. The history of Quinns Beach is characterised by a key challenge facing coastal communities – shoreline erosion and recession. Early reports of erosion date back to the 1940s and have continued through to present times. This ongoing erosion at Quinns Beach is a challenging issue that the City is keen to minimise and manage effectively.

The Quinns Beach Long Term Coastal Management Study was established by the City of Wanneroo with the goal of assessing potential coastal management options that would mitigate the ongoing trend of erosion present at Quinns Beach. The study area extends from the southernmost section of Quinns Beach, approximately 1 km north of the Mindarie Keys marina entrance, to the northernmost rocky outcrop approximately 2.2 km northwest of Groyne 3 (the northern-most existing groyne at Quinns Beach). This study area essentially includes all sandy coastlines from Mindarie Keys in the south to Alkimos Beach in the north, and includes several coastal protection structures, including the most recent (geotextile sand containers) works adjacent to Frederick Stubbs Park (Figure 1-1). To facilitate discussion Quinns Beach has been divided into six sections based on plan alignment, existing structures and historical shoreline changes, as depicted in Figure 1-1.

The overall study has been divided into five stages, namely

> Stage 1 – Undertake a detailed coastal processes assessment based on existing studies and recently collected data,

> Stage 2 – Assess coastal management options and identify a preferred option based on a multi-criteria analysis,

> Stage 3 – Provide detailed design drawings and technical specifications (suitable for tendering purposes) for the preferred coastal management option,

> Stage 4 – Provide technical advice during tendering and construction phases of the project,

> Stage 5 – Provide technical advice and coastal engineering services post-construction.

This report summarises Stage 3 of the Quinns Beach Long Term Coastal Management Study.

1.1 Selected Option

Following Stage 2 of the project (Cardno 2016), the two preferred options (labelled MCA2 and MCA3 in Cardno (2016) but referred to in the community engagement programme as Option 1 and Option 2, respectively) were presented to the Quinns Beach community during a comprehensive community engagement programme held in April/May 2016. As part of the process, the community were provided with the opportunity to submit feedback to the Council including a preference for Option 1 or Option 2. From the feedback received by the Council, the community preference was nearly evenly split between the two options.

The Council considered the Stage 2 report (Cardno 2016) and the question of which option to select to take to detailed design at the Council meeting held on 16 August 2016. The relevant agenda section as well as the relevant minutes section from this Council meeting are presented in Appendix A and detail the deliberations made by Council and the ultimate decision of selection of Option 1 (MCA2 in Cardno (2016)) as the option to take to Stage 3 (Detailed Design).

Based on the decision made at the Council meeting on 16 August 2016, the City requested Cardno to provide a quotation to proceed with the detailed design of Option 1 (MCA2 in Cardno (2016)). Cardno was formally commissioned to undertake the detailed design of Option 1 on 13 September 2016.

1.2 Stage 3 Scope of Work

The scope of work for Stage 3 of the Quinns Beach Long Term Coastal Management Study includes the following items:

> Geophysical Investigation,

> Coastal Modelling
  - Longshore sediment transport modelling for a 50-year timeframe,
  - Wave modelling to provide input to the 50-year sediment transport modelling,
  - Storm erosion modelling using the existing XBEACH model,
  - Extreme wave modelling for groyne structure design,
> Coastal Engineering Design
  - Detailed design of extensions to Groynes 2 and 3 and construction of new Groyne 4,
  - Analysis of capital and maintenance renourishment volumes,
  - Advice on potential future requirement for a Groyne 5, a seawall in front of the existing carpark and construction staging,
  - Monitoring and maintenance requirements,
> Concept level beach access design for Groyne 1, Groyne 4 and Waterland Point,
> Detailed design of a permanent access path for the new Groyne 4,
> Preparation of a Bill of Quantities (BoQ), P90 cost estimate and NPV analysis,
> Detailed design report,
> Technical drawings and specifications for issue to tender.

As per discussions with the City, the validation of the existing wave model to the additional measured wave data captured offshore of the Dog Beach during Stage 2 of the project, which was completed after submission of the Stage 2 report, is presented in Appendix C of this report. This data collection and validation of the existing wave model to this measured data was recommended in the Stage 1 report (Cardno 2015) based on the initial calibration results of the wave model in this area.

1.3 State Coastal Planning Policy Implications for Chosen Option

The WA State Planning Policy 2.6: State Coastal Planning Policy (SPP2.6) was introduced into legislation in 2006 to provide guidance for the long-term future planning of coastal communities in the recognition of the impacts of sea level rise and future threats to coastal habitats and infrastructure. Since the introduction of the legislation, it has taken some time for the development of the WAPC CHRMAP Guidelines (WAPC 2014) that outline a process for the development of a Coastal Hazard Risk Management and Adaptation Plan (CHRMAP). This process includes determining the value of coastal assets (both natural and built assets) and the use of these values to guide adoption of future planning strategies around four categories of Avoid, Retreat, Adapt and Protect. Whilst not explicitly stated in the guidelines, one of the key implications of SPP2.6 for local government is that future applications by local governments for State Funding of coastal issues will require a state-endorsed CHRMAP. The corollary is that local governments that do not adhere to the SPP2.6 guidelines and undertake coastal protection works without due consideration of future planning strategy will need to fund any future works themselves (including repair of coastal structures subject to future storm damage). Another key element in the implementation of the SPP2.6 guidelines is that significant community consultation be undertaken to ensure the local community (i.e. ratepayers) are informed of the implications of today's decisions to ensure that any adopted strategy appropriately considers inter-generational equity issues.

The City of Wanneroo has recognised the importance of SPP2.6 in guiding its future planning for the coastal zone and has embarked on a range of related studies. These include Part 1 of a CHRMAP (MPR 2015), Part 2 of a CHRMAP which is currently in progress, and a number of Quinns Beach coastal processes and erosion investigations undertaken since 1996. These investigations have culminated in this program of works at Quinns Beach which commenced in 2010. Each of these studies has incorporated significant community consultation, and, while generally the focus has been on short term (<10 years) planning issues, the more recent studies also consider the long term (50-100 years) planning within the SPP2.6 context. The ongoing beach erosion issue and adherence to the SPP2.6 guidelines demand different responses and timeframes often making it difficult to balance the requirements of the different demands. It should be noted that the area of Quinns Beach which is the subject of the Quinns Beach Long Term Coastal Management Study is currently excluded from the CHRMAP process due to the detailed investigations and modifications to the coastline in this area as a result of the project. Future revisions of the CHRMAP for the City of Wanneroo will include Quinns Beach after completion of the study and the resulting coastal protection works.

The current investigations and detailed design have considered the requirements of SPP2.6 as discussed at the Community Reference Group (comprising delegates representing community and State Agencies) meeting of 1 December, 2015 where the following was noted in the meeting minutes:

> *"State Coastal Planning Policy was discussed, specifically regarding protection options such as construction of a seawall in front of the existing carpark. Department of Planning commented that there should not be any issues with alignment with the State Policy for the proposed coastal protection options."*

At the Council meeting of 16 August 2016, it is also noted that:

> *"DoT Coastal Management supports the relocation of the Quinns car park to a less vulnerable location.*
>
> *The State Coastal Planning Policy requires coastal planning to adhere to the principles of coastal hazard risk management and adaptation planning (CHRMAP) by systematically identifying and understanding coastal hazard risks and putting in place controls to manage them. Coastal engineering works will only be supported if all other options for avoiding and adapting to coastal hazards have been fully explored, e.g. managed retreat in the form of relocation of the car park.*
>
> *The Department supports coastal engineering works to protect public assets only when the value of the public assets is greater than the cost of the works, i.e. for high value public infrastructure that is not expendable. The Department does not consider the existing car park to be a significant high value asset, and it is known to be located in the dynamic coastal zone. The Department considers the relocation of the car park a more sustainable option in the long term, as it is likely to maintain public beach access and a coastal foreshore reserve."*

Within the SPP2.6 framework the Council has supported a strategy of protecting the amenity and public infrastructure at Quinns Beach. The design of the groynes has included consideration of maintaining sufficient beach buffer to mitigate the effects of rising sea levels on the carpark over the design life of the groynes (50 years) and the requirements of SPP2.6 have been considered throughout the study.

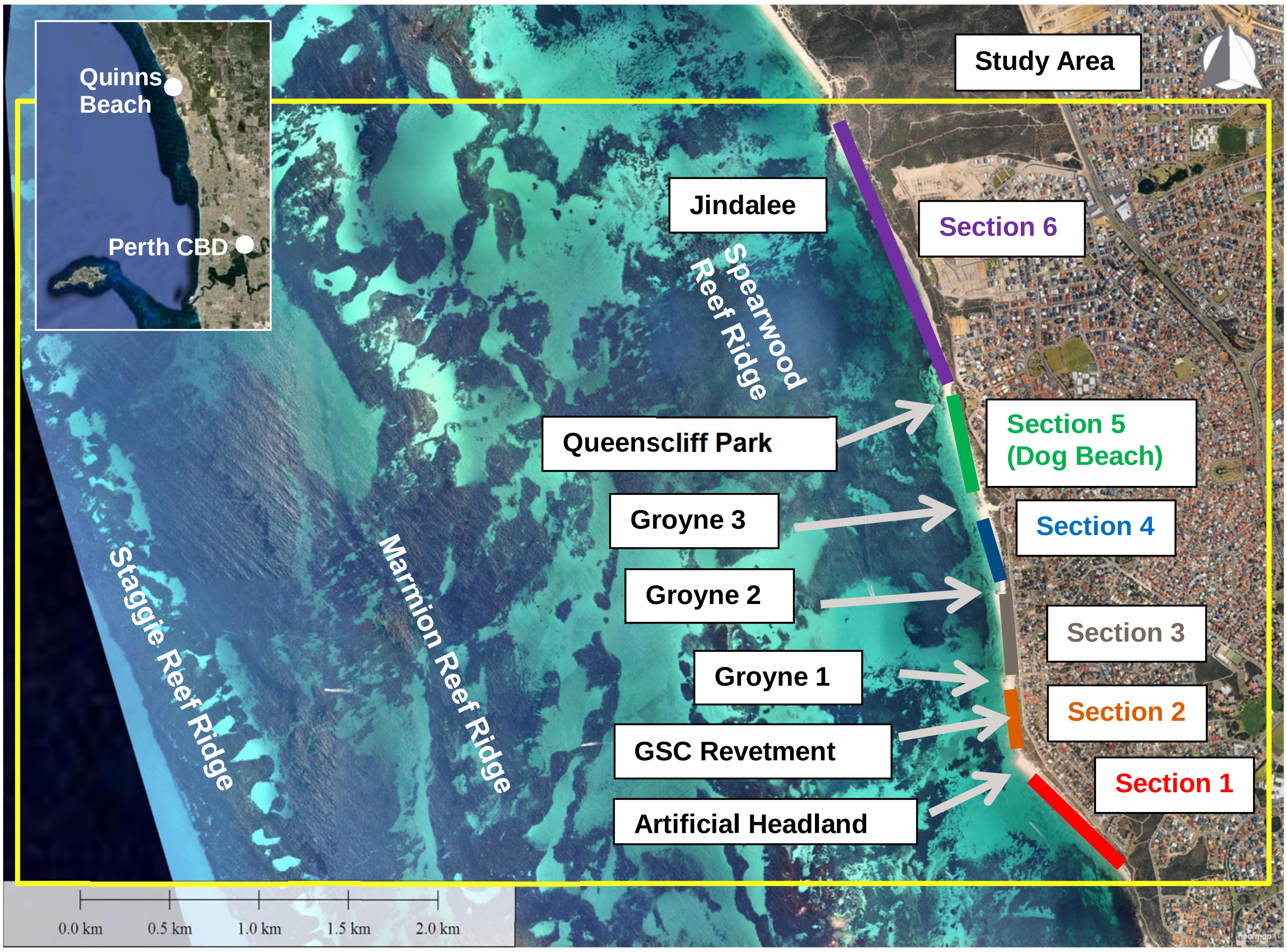

Figure 1-1 The Study Area; Quinns Beach within the City of Wanneroo, 35 km north of Perth

2 Geophysical Investigation

2.1 Introduction and Purpose

In accordance with the Stage 3 scope of work, a geophysical investigation of five different areas of Quinns Beach was commissioned by the City of Wanneroo and undertaken by GBG Maps Pty Ltd. The five areas that were investigated are shown in Figure 2-1.

Figure 2-1 Geophysical investigation areas (GBG Maps 2016)

The full GBG Maps report detailing the geophysical investigation is presented in Appendix B and the important observations and conclusions are summarised in this section. The purpose of the geophysical investigation was to investigate the presence (or absence) of hard substrate beneath the beach and dunes that might act as a physical barrier to potential future coastal erosion in each of the areas surveyed.

The geophysical investigation consisted of a series of both cross-shore and alongshore transects using a combination of subsurface test methods (Multi-Channel Analysis of Surface Waves (MASW) and Seismic Refraction). Cone Penetrometer Testing (CPT) was also conducted to provide 'ground-truthing' and to calibrate the geophysical results. Refer to the GBG MAPS report in Appendix B for a full description of the instrumentation and methodology applied in the investigation.

2.2 Overall Summary

This section provides a general summary of the results of the investigation and the subsequent sections will summarise the results of each specific section that was investigated.

While the geophysical investigation only provides results as vertical cross-sections directly underneath each transect, the combination of alongshore and cross-shore transects gives an indication of the likely general conditions in the area bounded by the lines in each area investigated.

As a generalised summary, the geophysical investigation found the following:

1. All the transects consist of either sand of low compaction, sand of moderate compaction, limestone of interpreted low-moderate rock strength and limestone of interpreted moderate rock strength. No other material types or layers were found during the investigation.
2. Limestone of interpreted low-moderate rock strength was found at some depth along the full length of all transects showing that nowhere along any of the transects did the substrate consist entirely of sand.
3. All the cross-shore transects show a similar profile shape that mirrors the surface profile of the beach and dune landforms, which consist of the top of the interpreted low to moderate strength limestone layer being close to mean sea level (MSL) at the seaward end of the transects and gradually increasing in elevation landward along the length of the transect.
4. All the alongshore transects showed the layer of interpreted low-moderate strength limestone at generally consistent elevations along the length of each alongshore transect.
5. The lowest elevation of the top of the layer of limestone of interpreted low-moderate strength at the landward end of a cross-shore transect is approximately +2.5m AHD which is approximately 2.5m above current MSL and approximately 0.7 metres above the 100-year ARI extreme water level of +1.68m AHD (which includes an allowance for sea level rise over the 50 year design life of the structures of 0.4m). This lowest elevation occurs on cross-shore transect 1-1, which is located at the western end of the caravan park site in the southern part of Section 1 of the project area.
6. The highest elevation of the top of the layer of limestone of interpreted low-moderate strength at the landward end of a cross-shore transect is approximately +13m AHD which is approximately 13m above current MSL and 11.3 metres above the 100-year ARI water level of +1.68m AHD (which includes an allowance for sea level rise over the 50 year design life of the structures). This occurs on cross-shore transects 3-1, 4-2, and 4-3 which are located just north of Groyne 3, at the location of the existing staircase at Queenscliff Park, and approximately 150m further north at approximately the mid-point between the proposed location of Groyne 4 and the beach access at Waterland Point, respectively.
7. The main conclusion to be drawn from the results of the geophysical investigation is that the presence of hard substrate underneath the beach and dune system across the various areas of Quinns Beach that were investigated showed that the limestone of interpreted low-moderate strength, which was located beneath the ground level, is likely to provide some level of protection in terms of acting as a physical barrier to coastal erosion; though the degree of protection is likely to be quite variable between the different areas surveyed. This variability is due to multiple causes, such as the variability in the elevation of the limestone layer in both the cross-shore and alongshore directions, the height and width of the dune system between the current beach face and any infrastructure located behind it and the depth of sand overlying the limestone layer(s).
8. Of the areas surveyed, two showed the limestone layer at lower elevations than the others and can thus be considered as having a hard substrate, which is likely to provide a lower level of protection

from coastal erosion in terms of acting as a physical barrier to coastal erosion. These areas are north of Groyne 2 and north of Groyne 3. The area north of Groyne 2 can be considered to be more vulnerable, mainly due to the closer proximity of infrastructure to the current shoreline. These areas should be paid particular attention during monitoring in the post-construction period and during planning of annual renourishment requirements.

2.3 Site 1: Southern Part of Section 1 (Caravan Park)

At this location two cross-shore transects, one alongshore transect, and two CPT pits were completed as part of the geophysical investigation, the locations of which are shown in Figure 2-2. Refer to the GBG Maps report in Appendix B for more detailed information and plots of the results along each transect and at each CPT test pit.

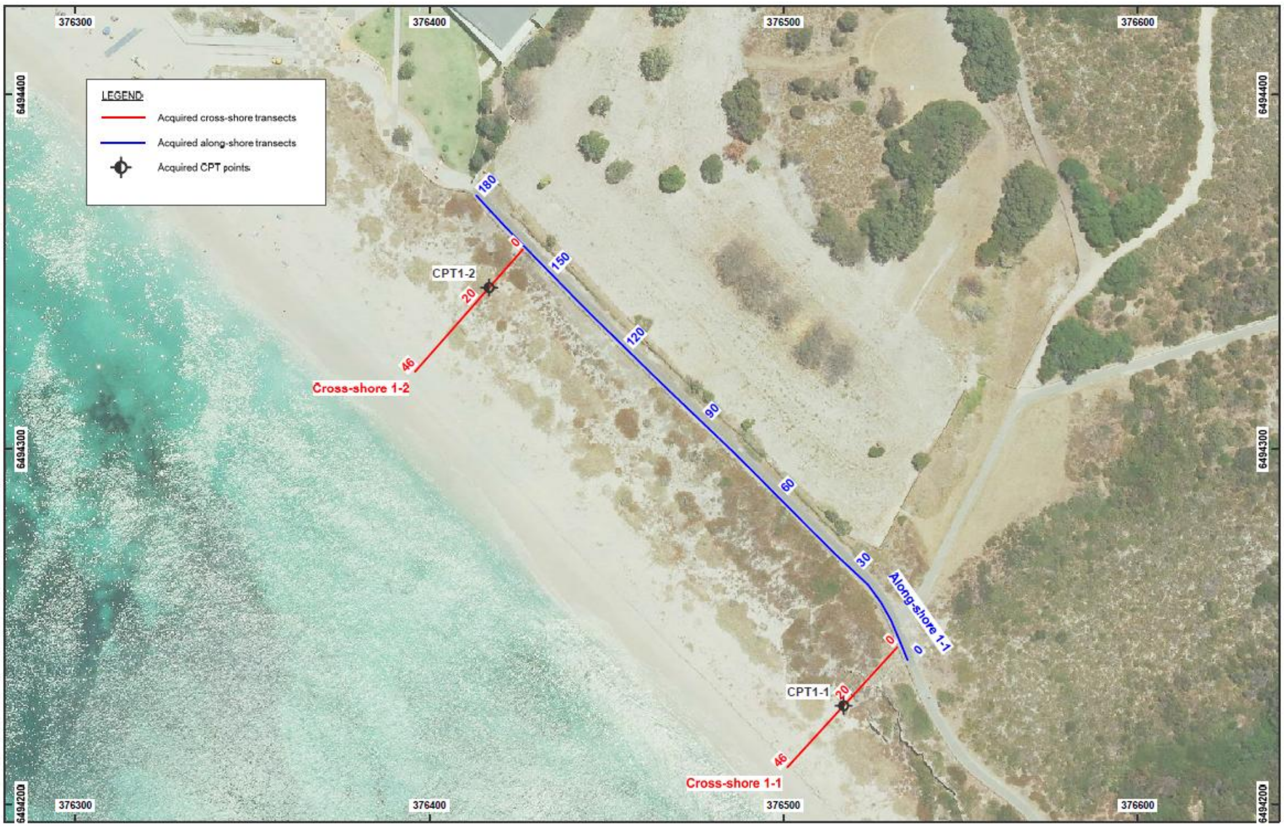

Figure 2-2 Locations of cross-shore (red) and alongshore (blue) transects and CPT pits completed at Site 1 (Caravan Park) (GBG Maps 2016)

A summary of the observations and conclusions from the geophysical investigation results in this area is:

1. Cross-shore transect 1-1 (eastern cross-shore transect) shows the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and approximately +6.5m AHD at the landward end of the transect, which is adjacent to the bitumen path.
2. Cross-shore transect 1-2 (western cross-shore transect) shows that the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and approximately +3m AHD at the landward end of the transect which is adjacent to the bitumen path.
3. Alongshore transect 1-1 (located immediately seaward of the bitumen path which runs perpendicular to the shoreline) shows that the top of the low-moderate strength limestone layer varies between approximately +2m AHD in the western part of the transect and approximately +7m AHD in the eastern part of transect.
4. The main observation to be drawn from the geophysical results in this area is that the beach and dune in front of this section of the coast contains underlying limestone layers and that these layers reach elevations of between +2 and +7m AHD underneath the top of the dune adjacent to the bitumen path.

The elevation of the current surface (generally sand) level along this transect is approximately +7 to +8m AHD, which is between approximately 0 and 5 metres above the underlying limestone layer.

5. The main conclusion to be drawn from the results in this area is that, should future coastal erosion occur along this section of the coast, the limestone present underneath the dune will likely provide some protection in terms of acting as a physical barrier to coastal erosion, though the degree of protection is likely to be variable along this beach section. The dune elevation and underlying limestone layers along the seaward edge of the bitumen path at the back of the beach in this area are lower than the other areas investigated as part of this study and the areas where the limestone is at elevations higher than extreme water levels at the site are located towards the back of the beach and relatively close to the bitumen path. Should significant erosion occur in this area in the future, it is possible the bitumen path may be threatened or damaged by these coastal erosion processes and so future planning should take this into account. In particular, the results show the limestone is at higher elevations (and hence should provide greater protection) in the eastern part of this section of beach and lower elevations (and hence lower protection) in the western part. The fact that the natural surface level (generally sand) is relatively close (<5m higher) to the underlying limestone layer means that, should coastal erosion occur, then the erosion scarp that would form at the dune face as the beach and dune was eroded would not extend as far landward as it would if the underlying rock level at the top of the dune were significantly lower.

2.4 Site 2: Groyne 2

At this location three cross-shore transects and three CPT pits were completed as part of the geophysical investigation, the locations of which are shown in Figure 2-3. Refer to the GBG Maps report in Appendix B for more detailed information and plots of the results along each transect and at each CPT test pit.

Figure 2-3 Locations of cross-shore transects (red) and CPT pits completed at Site 2 (Groyne 2) (GBG Maps 2016)

A summary of the observations and conclusions from the geophysical investigation results in this area is:

1. Cross-shore transect 2-1 (southern cross-shore transect) shows the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and

approximately +5m AHD at the landward end of the transect which is adjacent to the limestone wall/fence and footpath immediately seaward of Ocean Drive.

2. Cross-shore transect 2-2 (middle cross-shore transect) shows the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and approximately +6.5m AHD at the landward end of the transect which is adjacent to the limestone wall/fence and footpath immediately seaward of Ocean Drive.
3. Cross-shore transect 2-3 (northern cross-shore transect closest to timber staircase) shows the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and approximately +3.5m AHD at the landward end of the transect which is adjacent to the limestone wall/fence and footpath immediately seaward of Ocean Drive.
4. The main observation to be drawn from the geophysical results in this area is that the beach and dune along this section of coast north of Groyne 2 contains underlying limestone layers and that these layers reach elevations of between +3.5m and +6.5m AHD at the landward end of the transects underneath the dune immediately seaward of the limestone wall/fence and footpath adjacent to Ocean Drive. The elevation of the current surface (sand) level at these same points is approximately +11 to +14m AHD which is consistently approximately 7.5 metres above the underlying limestone layer.
5. The main conclusion to be drawn from the results in this area is that, should future coastal erosion occur along this section of the coast, the limestone present underneath the dune will likely provide limited protection in terms of acting as a physical barrier to coastal erosion, and the degree of protection is likely to be variable along this beach section. Additionally, it is possible coastal erosion in this section of Quinns Beach could have a large impact on the beach and dune face (and potentially the infrastructure behind it including Ocean Drive) due to how much higher the current sand surface level is above the underlying limestone layer. When subject to erosion, the dune face will form an erosion scarp which will be a slope and so the erosion would not have to physically reach the location of infrastructure to start to undermine and damage it.
6. Based on the results in this section it is recommended that this section of Quinns Beach is paid particular attention during monitoring and when deciding which areas to renourish at a particular point in time, in the knowledge that the limestone substrate in this area is likely to provide only limited protection from coastal erosion, should it occur.

2.5 Site 3: Groyne 3

At this location one cross-shore transect and one CPT pit were completed as part of the geophysical investigation, the locations of which are shown in Figure 2-4. Refer to the GBG Maps report in Appendix B for more detailed information and plots of the results along the transect and at the CPT test pit.

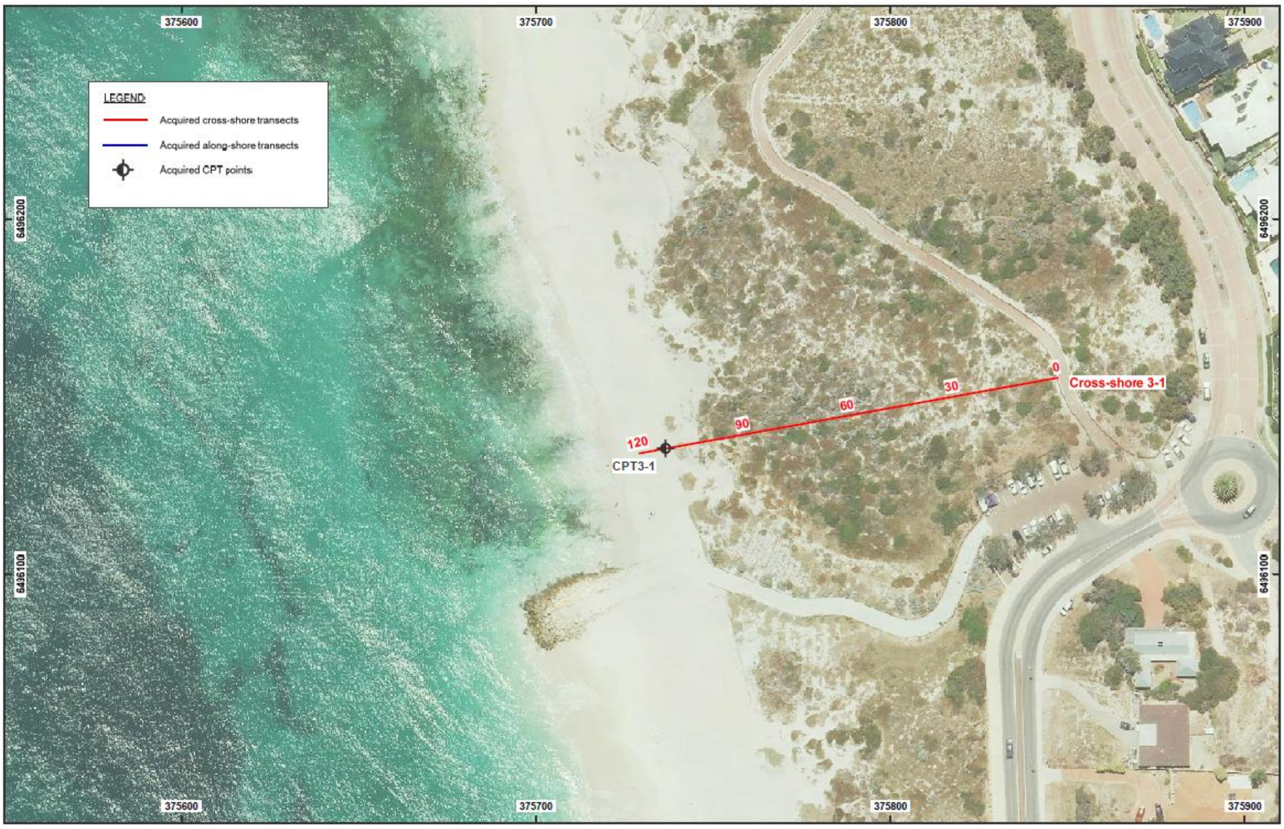

Figure 2-4 Locations of cross-shore transect (red) and CPT pit completed at Site 3 (Groyne 3) (GBG Maps 2016)

A summary of the observations and conclusions from the geophysical investigation results in this area is:

1. Cross-shore transect 3-1 shows the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and approximately +13m AHD at the landward end of the transect which is adjacent to the pedestrian path as shown in Figure 2-4.
2. The main observation to be drawn from the geophysical results in this area is that the beach and dune just north of Groyne 3 contain underlying limestone layers and that these layers reach elevations of approximately +13m AHD at the landward end of the transect underneath the dune immediately seaward of the pedestrian path as shown in Figure 2-4. The elevation of the current surface (sand) level at this same point is approximately +20m AHD which is approximately 7 metres above the underlying limestone layer.
3. The main conclusion to be drawn from the results in this area is that, should future coastal erosion occur along this section of the coast, the limestone present underneath the dune will likely provide substantial protection to the infrastructure in the area in terms of acting as a physical barrier to coastal erosion, however the degree of protection is likely to be variable along this beach section. The important infrastructure in this area north of groyne 3 is all setback further from the current coastline and also the dunes in this area are substantially larger and higher than in the areas further south providing further buffer against coastal erosion. However, it is possible coastal erosion in this section of Quinns Beach could have a large impact on the beach and dune face due to how much higher the current sand surface level is above the underlying limestone layer. When subject to erosion, the beach and dune face will form an erosion scarp which will be a slope and would likely result in reduced beach amenity in the area.
4. Based on the geophysical results in this section and the fact that the Quinns Beach community places a high value on this section of Quinns Beach as it is the southern end of the dog beach, it is recommended that this area be closely monitored post-construction, in particular during early winter when the natural seasonal trends are expected reduce the beach width immediately north of Groyne 3 to its minimum annual width. The condition of this section of Quinns Beach should be considered

when deciding annual renourishment requirements, particularly if the renourishment is to occur in later summer/early winter.

2.6 Site 4: Proposed Groyne 4

At this location four cross-shore transects, two alongshore transects and four CPT test pits were completed as part of the geophysical investigation, the locations of which are shown in Figure 2-5. Refer to the GBG Maps report in Appendix B for more detailed information and plots of the results along the transect and at the CPT test pit.

Figure 2-5 Locations of cross-shore (red) and alongshore (blue) transects and CPT pits completed at Site 4 (Proposed Groyne 4) (GBG Maps 2016)

A summary of the observations and conclusions from the geophysical investigation results in this area is:

1. Cross-shore transect 4-1 shows the top of the low-moderate strength limestone layer varies between approximately -2m AHD at the seaward end of the transect and approximately +10m AHD at the landward end of the transect which is adjacent to one of the footpaths in Queenscliff Park as shown in Figure 2-5.
2. Cross-shore transect 4-2 shows the top of the low-moderate strength limestone layer varies between approximately +1m AHD at the seaward end of the transect and approximately +12.5m AHD at the landward end of the transect which is adjacent to the current pedestrian access path down to the stairs as shown in Figure 2-5.
3. Cross-shore transect 4-3 shows the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and approximately +13m AHD at the landward end of the transect which is adjacent to the footpath and Ocean Drive.
4. Cross-shore transect 4-4 shows the top of the low-moderate strength limestone layer varies between approximately -2m AHD at the seaward end of the transect and approximately +10m AHD at the landward end of the transect which is adjacent to the footpath and carpark at Waterland Point.

5. Alongshore transect 4-1 (located in Queenscliff Park as shown in Figure 2-5) shows that the top of the low-moderate strength limestone layer varies between approximately +11m AHD in the southern part of the transect and approximately +13.5m AHD in the northern part of transect.
6. Alongshore transect 4-2 (located north of alongshore transect 4-1 and along the seaward edge of the footpath next to Ocean Drive and Waterland Point) shows that the top of the low-moderate strength limestone layer varies between approximately +7m AHD in a small section approximately 30m long in the southern part of the profile and approximately +17m AHD in the northern part of transect.
7. The main observation to be drawn from the geophysical results in this area is that the beach and dune between the position of the proposed Groyne 4 and the beach access at Waterland Point contain underlying limestone layers and that these layers reach elevations of approximately +7 to +17m AHD underneath the dune along the two alongshore transects as shown in Figure 2-5. The elevation of the current surface (sand) level along these same two transects is approximately +17 to +21m AHD which is between approximately 5.5 to 11.5 metres above the underlying limestone layer.
8. One specific result should be highlighted in this area, which is a section of alongshore transect 4-2 (approximately between distances 50 and 80m along the profile) which shows the underlying limestone layer to be at least 5m lower than the area either side of it. This section is close to the part of this section of Quinns Beach where Ocean Drive is closest to the current shoreline. The elevation of the limestone layer in this section is around +7m AHD which still significantly above normal and even extreme water levels.
9. The main conclusion to be drawn from the results in this area is that, should future coastal erosion occur along this section of the coast, the limestone present underneath the dune will likely provide substantial protection to the infrastructure in the area in terms of acting as a physical barrier to coastal erosion, however the degree of protection is likely to be variable along this beach section. The dunes in this area are substantially larger and higher than in the areas further south providing a greater buffer against coastal erosion. However, it is possible coastal erosion in this section of Quinns Beach could have a large impact on the beach and dune face due to how much higher the current sand surface level is above the underlying limestone layer. When subject to erosion, the beach and dune face will form an erosion scarp which will be a slope and would likely result in reduced beach amenity in the area.
10. Based on the geophysical results in this section and the fact that the Quinns Beach community places a high value on this section of Quinns Beach as it is the northern half of the dog beach, it is recommended that this area be closely monitored post-construction, in particular during early winter when the natural seasonal trends are expected reduce the beach width immediately north of Groyne 4 to its minimum annual width. The condition of this section of Quinns Beach should be considered when deciding annual renourishment requirements, particularly if the renourishment is to occur in later summer/early winter.

2.7 Site 5: Northern Area

At this location one cross-shore transect, one alongshore transect and three CPT test pits were completed as part of the geophysical investigation, the locations of which are shown in Figure 2-6. Refer to the GBG Maps report in Appendix B for more detailed information and plots of the results along the transect and at the CPT test pit.

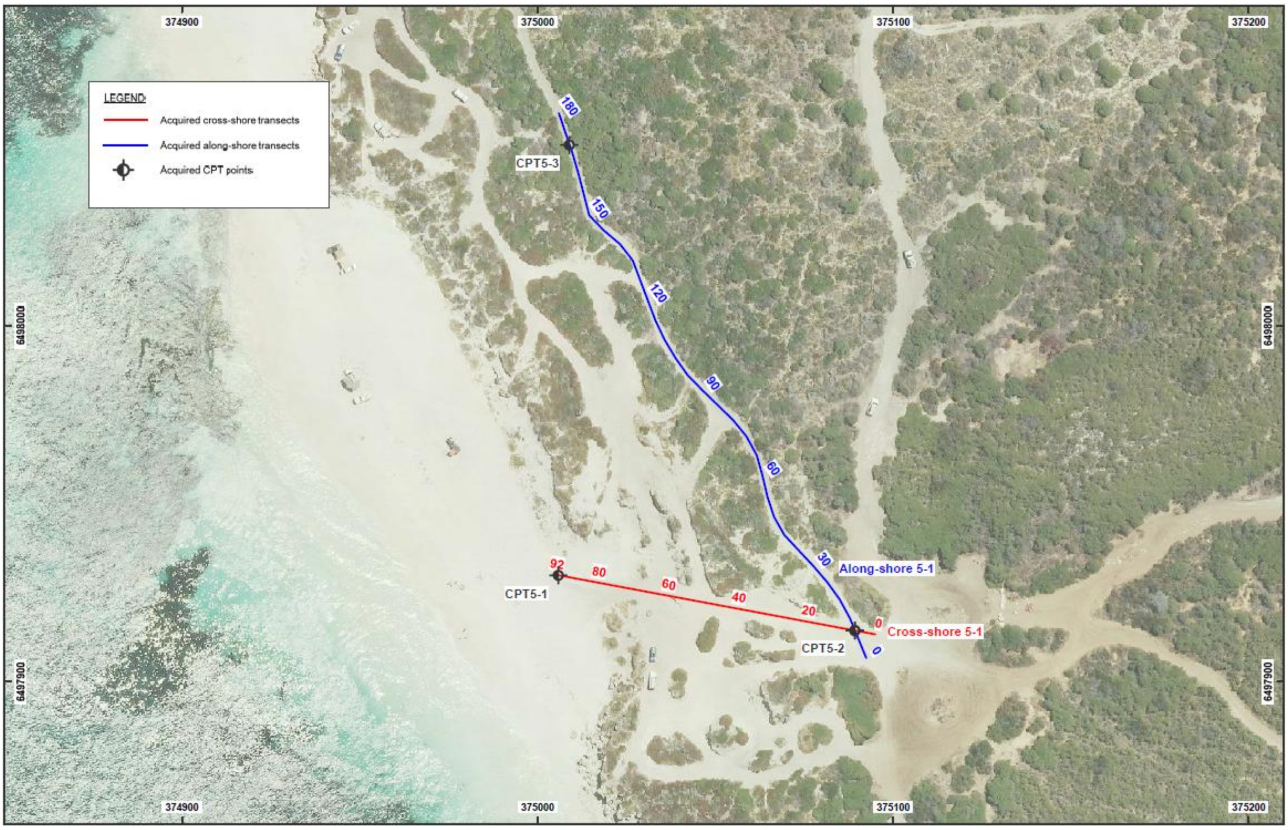

Figure 2-6 Locations of cross-shore (red) and alongshore (blue) transects and CPT pits completed at Site 5 (Northern Area) (GBG Maps 2016)

A summary of the observations and conclusions from the geophysical investigation results in this area is:

1. Cross-shore transect 5-1 shows the top of the low-moderate strength limestone layer varies between approximately 0m AHD (MSL) at the seaward end of the transect and approximately +9.5m AHD at the landward end of the transect which is as shown in Figure 2-6.
2. Alongshore transect 5-1 (located along a 4WD track as shown in Figure 2-6) shows that the top of the low-moderate strength limestone layer varies between approximately +4.5m AHD in the northern part of the transect and approximately +10m AHD in the mid-southern parts of transect.
3. The main observation to be drawn from the geophysical results in this area is that the beach and dune in the northern area contain underlying limestone layers and that these layers reach elevations of approximately +4.5 to +10m AHD underneath the dune along the alongshore transect as shown in Figure 2-6. The elevation of the current surface (generally sand) level along this same transect is approximately +10 to +13m AHD which is between approximately 3 and 5.5 metres above the underlying limestone layer.
4. The main conclusion to be drawn from the results in this area is that, should future coastal erosion occur along this section of the coast, the limestone present underneath the dune will likely provide substantial protection to the existing dune system in terms of acting as a physical barrier to coastal erosion, however the degree of protection is likely to be variable along this beach section. However, it is possible coastal erosion in this section of Quinns Beach could have an impact on the beach and dune face due to how much higher the current sand surface level is above the underlying limestone layer. When subject to erosion, the beach and dune face will form an erosion scarp, which will be a slope and would likely result in reduced beach amenity in the area. There is currently no infrastructure in this area of Quinns Beach, however, any future development should take this risk into account during the planning process.

3 Longshore Sediment Transport Assessment

3.1 Introduction

A longshore sediment transport and beach plan-alignment change assessment was undertaken to further refine and optimise the groyne lengths for the chosen option (MCA2\LITPACK Option 12 in Cardno (2016)). For simplicity, this layout is herein referred to in this section of this report as “Option 12” for consistency when compared with other layout options simulated in LITPACK.

Option 12 in Cardno (2016) consisted of the following:

> Extension of Groyne 2 from approximately 55m to 120m length,

> Extension of Groyne 3 from approximately 55m to 75m length,

> Construction of new Groyne 4 to 60m length.

This longshore sediment transport assessment was undertaken using the LITPACK model which was developed, calibrated and validated during the previous stages of the project (Cardno 2015, 2016). As described in Cardno (2015), the initial bathymetry applied in the LITPACK model is the measured survey from May 2008. The LITPACK simulations incorporate the capital renourishment which is included in the selected option but not the annual renourishment which is intended to be undertaken each year. The LITPACK model has been extended to simulate a period of 50 years to assist in identifying long term shoreline movement trends for different refinements of the chosen option. All LITPACK model runs carried out in this stage of the project were for the 50-year period as described above.

The derivation of the 50-year wave climate used to force the LITPACK model is outlined in Appendix C. This climate was derived by repeating the 10-year wave climate applied in the previous stages of the project, but incorporating sea level rise and associated increased nearshore wave heights over the course of the 50-year simulation. This ‘nose-to-tail’ technique is used commonly where very long periods of wave data are not available. The wave model applied for the 50-year simulations was re-validated using the additional data captured at the dog beach location during Stage 2 of the project as recommended in Cardno (2015). The outcome of the re-validation was that a constant bed roughness map was applied in the wave model when applied to produce forcing conditions for sediment transport while a variable bed roughness map was applied where the wave model was applied to produce extreme design criteria (refer Appendix C for details).

The LITPACK assessment was undertaken following these stages:

1. Simulate the existing groyne layout for 50 years to provide a base case for comparison,
2. Simulate the chosen layout option (Option 12 which is summarised in Figure 3-1) for 50 years to provide a benchmark for optimisation of the layout,
3. Refinement of the Groyne 2 length by simulating varying lengths of Groyne 2, with Groynes 3 and 4 set as per Option 12.
4. Refinement of the Groyne 3 length by simulating varying lengths of Groyne 3, using the refined Groyne 2 length, and Groyne 4 as per Option 12.
5. Refinement of the Groyne 4 length by simulating varying lengths of Groyne 4, with Groynes 2 and 3 as per the refined lengths determined during steps 3 and 4.
6. Refinement of the Groyne 4 location along Quinns Beach by simulating varying locations of Groyne 4.

To avoid confusion when referring to the previous stages of the project, the simulation nomenclature used in this report follows on from the Stage 2 study. The modelled groyne layouts start with Option 12 (being the final option presented in the Stage 2 study, which also happens to be the layout chosen to take to detailed design), and the options developed during the course of this stage of the project are sequenced as Option 13 onwards.

In all longshore sediment transport result plots, a positive value for coastline or shoreline change indicates accretion and a negative value indicates erosion.

Figure 3-1 MCA2/LITPACK Option 12 from Cardno (2016). Groyne 1 unchanged in length, Groynes 2 and 3 extended. Groyne 4 new structure.

3.2 Longshore Transport Model Limitations and Uncertainties

Numerical models inherently include assumptions and simplifications in order to model the complex natural world. The limitations and uncertainties of the numerical models utilised in this study are discussed in detail in section 7.1.3 of this report. The LITPACK longshore transport model developed for Quinns Beach was calibrated with a focus on the most important areas of interest along Quinns Beach. Some of the model projections presented in the following sections for the areas of Quinns Beach near the boundaries of the model domain (Sections 1 and 6) differ from recent observed trends (Section 1) or expectations of the influence of new coastal structures based on previous experience at other locations in the Perth Metropolitan area (Section 6). As such the model projections over the long-term for these sections of Quinns Beach should treated with caution.

3.3 Groyne 2 Length Assessment

3.3.1 Description of Groyne 2 Options

The Option 12 layout selected by the City following Stage 2 of the study included a 60m long extension to the existing Groyne 2 making the total length 120m. To optimise the length of Groyne 2, two additional LITPACK simulations were undertaken, these were:

> **Option 13 –** Extending groyne 2 by only 30m from the existing length such that the total length of Groyne 2 was 90m,

> **Option 14 –** Extending groyne 2 by only 45m from the existing length such that the total length of Groyne 2 was 105m.

These options are presented graphically in Figure 3-2, where the groyne lengths for the existing as well as Options 12, 13 and 14 are shown.

3.3.2 Results

The LITPACK results are presented in this section. Given the inter-annual and seasonal variability of the wave climate at the site, and the relatively small annual transport rates, a number of plots have been produced for each simulation. They include:

1. 1D plots showing the difference between the average shoreline position during the last decade of the simulation period for each option and the average shoreline position of the existing groyne layout after the model has stabilised (years 10 to 20);
2. Time series plots of the average beach width in each section over the 50-year simulation. The beach width was calculated by determining the position of the 0m AHD contour line relative to a baseline and finding the average distance between the two lines for each time step. The shoreline within 20m of a modelled structure was not included in the calculation in order to exclude the significant variability around each structure and to focus on the overall trend of the shoreline within each section of Quinns Beach.
3. Maps of the average, maximum and minimum shoreline positions during the last 10 years of the simulation period overlaid on an aerial image for each section of Quinns Beach (refer to Appendix D);

Results are discussed below with each layout compared to the existing case simulation. Reference is also made to the modelling undertaken during the previous stages of the project to highlight the effects of the changes to the structure layouts.

Figure 3-2 Overview of the Groyne 2 Layouts simulated in the LITPACK model

Figure 3-3 and Figure 3-4 present a summary of the modelling results for the Groyne 2 length refinement. As noted above, these present the relative shoreline change predicted by the model along the beach and time series plots of the average shoreline position over the simulation period, respectively. Based on these results the model is predicting that altering the length of Groyne 2 will have the most influence on the beach position between Groynes 1 and 3 (Sections 3 and 4 in Figure 1-1); however some differences in other sections of beach are also predicted.

Section 1

> In Section 1 the model is predicting a generally erosive trend for both the existing and proposed layouts. A retreat of the shoreline of up to 15m at the end of the 50-year simulation is projected. This goes against recent observed trends in this section and so the long-term model projections in this section should be treated with caution.

> Of the Groyne 2 length options simulated, a 60m extension is predicted to retain slightly more sand compared to the other options, however this is only evident in the last 20 years of the simulation period and is of the order of 2-5m.

> A 45m extension of Groyne 2 is predicted to have only marginal influence on the beach in this section, with the model predicting the difference in shoreline position between the existing layout and the 45m extension of Groyne 2 is less than 1m at the end of the 50-year simulation.

> A 30m extension of Groyne 2 is predicted to result in slightly less beach width compared to the existing case, where the model is predicting that this option would have up to 2m less width in Section 2 after 50 years compared to the existing layout.

Section 2

> Similar trends are predicted in Section 2 with the model indicating very little difference in shoreline position between the options. The model indicates that the average beach width in Section 2 will vary seasonally by up to 10m.

> A slightly erosive trend is predicted in Section 2 with the current layout. The average shoreline recession in this section for the existing groyne layout is predicted to be around 2m over 50 years.

> Extending Groyne 2 by 60m is predicted to retain an additional 2-5m of beach width on average compared to the existing groyne layout.

> Extending Groyne 2 by 45m would have a marginal influence on the beach width in this section compared to the existing layout.

> The model predicts that if Groyne 2 is extended by 30m the beach will recede slightly compared to the existing layout.

Section 3

> The length of Groyne 2 has a marked effect on the width of the beach between Groynes 1 and 2. The model is predicting that the existing groyne layout would result in a relatively steady recession of about 1m every 3 to 4 years. At the end of the 50-year simulation the model is indicating the average shoreline position would have receded by 15m.

> The model indicates that by extending Groyne 2 by 30m the beach position in this section would stabilise, with little to no recession or accretion predicted over the 50-year simulation period.

> Extending Groyne 2 by 45m is predicted to result in an accreted beach, with the average beach width predicted to be between 5 and 10m wider than the initial position. This is predicted throughout the 50 year simulation.

> Extending Groyne 2 by 60m would result in a slightly larger average beach width (2-3m on average), compared to the 45m extension.

Section 4

> The length of Groyne 2 is predicted to have a significant effect on the beach width in Section 4. For the existing layout, the model is predicting a general accretion of around 10m in Section 4 for the first 20 years of the simulation, then recession thereafter back to the starting position.

> Extending Groyne 2 by 30m is predicted to result in an accretion of around 10m to 15m over the 50-year simulation period.

> A 45m extension of Groyne 2 is predicted to result in a relatively stable beach width, which is predicted to be 2-3m wider than the initial beach width.

> The model predicts that the beach width in Section 4 will reduce by around 7m if Groyne 2 is extended by 60m.

Section 5

> The length of Groyne 2 has only a slight influence on the beach width in Section 5. The model indicates that there is little to no difference in the beach width for a 30 or 45 m extension.

> A 60m extension of groyne 2 is predicted to result in a slightly narrower width of beach in section 5, however this is still predicted to be wider than the existing beach.

Section 6

> The length of groyne 2 has only a slight influence on the beach width in section 6 (north of groyne 4). The model indicates that there is little to no difference in the beach width for a 30 or 45 m extension.

> A 60m extension of Groyne 2 is predicted to result in a slightly narrower width of beach in Section 6, however, this is still predicted to be wider than the existing beach.

> The long-term accretive trend in this section is counter to expectations based on previous experience at other locations and so the long-term model projections in this section should be treated with caution. However, the model should be able to predict the relative change in shoreline position between the options.

3.3.3 Preferred Length of Groyne 2

Based on the modelling results, and after discussions with the City and DoT the preferred extension length of Groyne 2 is 45m. This is due to:

> The critical sections of Quinns Beach are Sections 3 and 5. A 45m extension of Groyne 2 provides a stable and wider beach in these sections without causing any significant erosion in other areas of Quinns Beach;

> Extending the groyne by 60m is predicted to result in narrower beach widths north of Groyne 2, with significant erosion noted in Section 4;

> Extending Groyne 2 by 30m results in less sand in Section 3, as well as slightly more erosion in Sections 1 and 2.

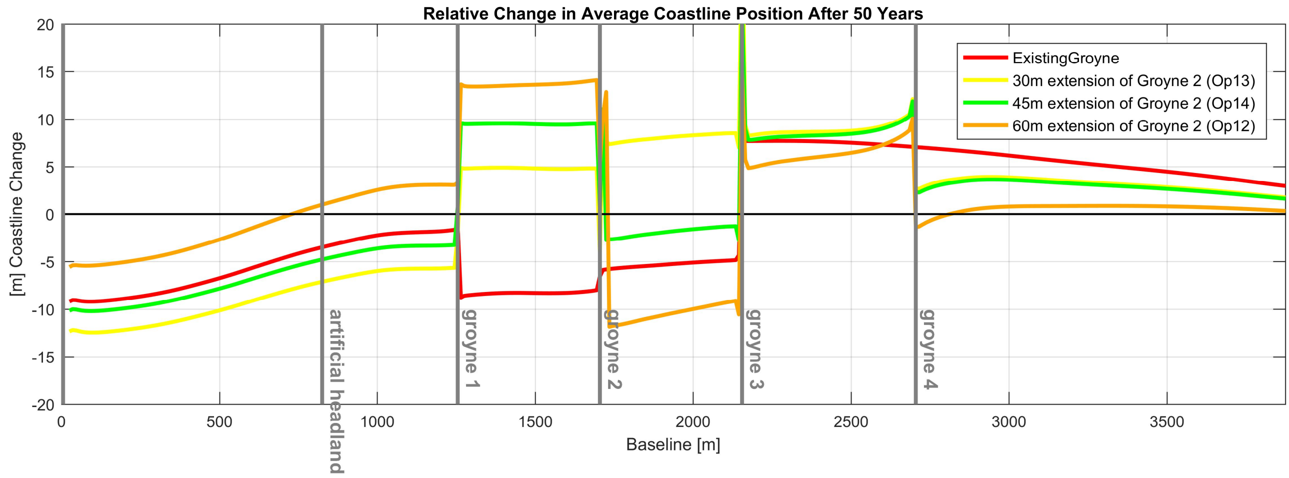

Figure 3-3 1D Plot showing Average Shoreline Position during the Last Decade of Simulation for Options 12, 13 and 14 relative to that of the Existing Groyne Layout after Model has stabilised (years 10 to 20)

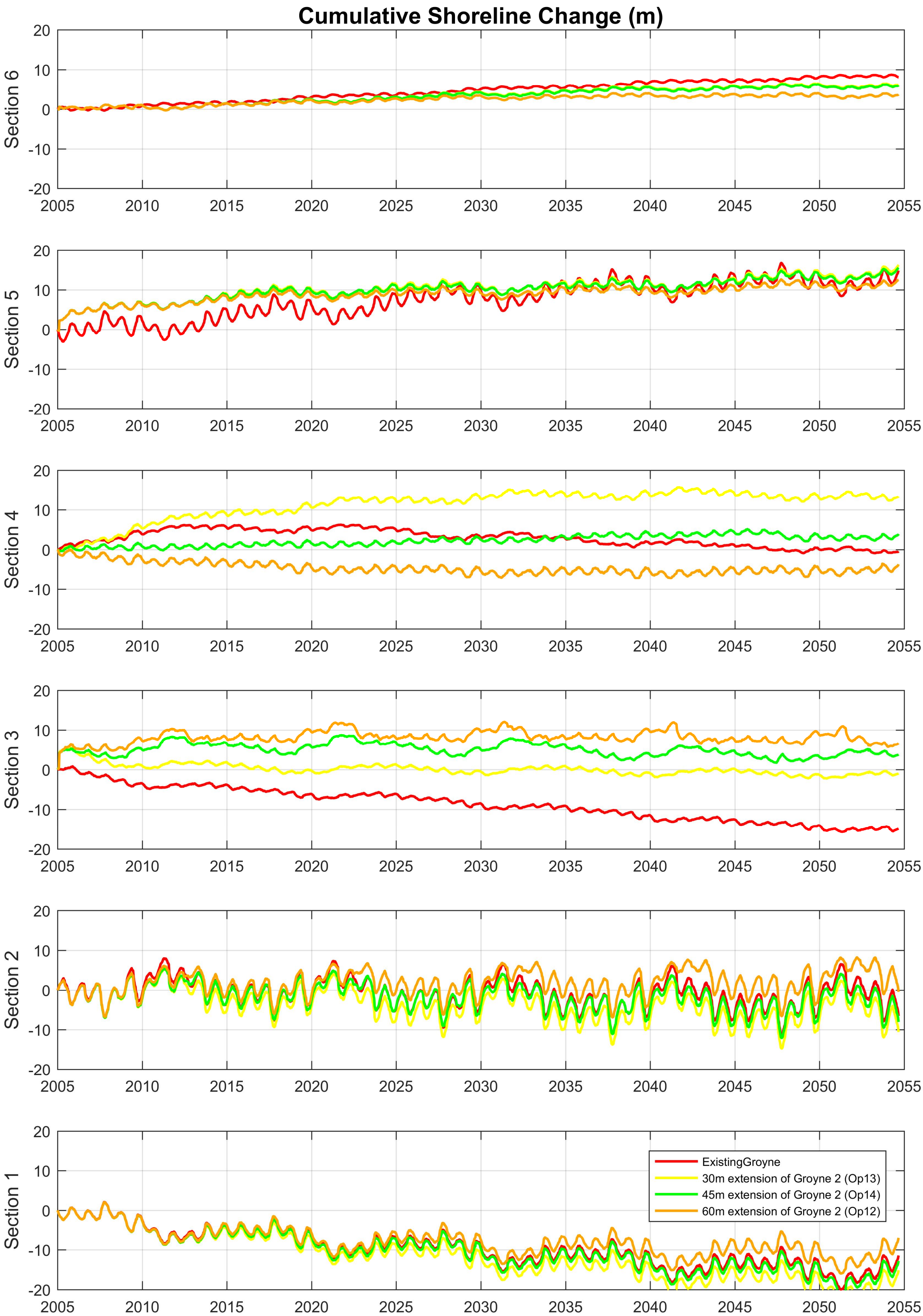

Figure 3-4 Time Series Plots of the Average Beach Width in each Section over the 50-year Simulation for Options 12, 13 and 14

3.4 Groyne 3 Length Assessment

3.4.1 Description of Options

Additional LITPACK simulations were completed to optimise the length of Groyne 3. They included:

> **Option 15 –** No extension of Groyne 3 from the existing layout, leaving the groyne length as 55m.

> **Option 14 –** Extending Groyne 3 by 20m from the existing length. The total length of Groyne 3 in this option is 75m.

> **Option 16 –** Extending Groyne 3 by another 10m. The total length of Groyne 3 in this option is 85m.

These options are presented graphically in Figure 3-5, where the groyne lengths for the existing and Options 14, 15 and 16 is shown.

Figure 3-5 Overview of the Groyne 3 Layouts simulated in LITPACK

3.4.2 Results

The LITPACK results are presented below in Figure 3-6 and Figure 3-7. These figures show the relative effect of the groynes on the average shoreline position over time. Figure 3-6 shows the relative change in the average beach width along the shoreline, whereas Figure 3-7 presents a time series of the average beach width in each section throughout the 50-year simulation period.

The results are discussed below in terms of each section of Quinns Beach.

Section 1

> The length of Groyne 3 does not appear to have any significant effect on the beach in Section 1. The model does not predict any discernible difference in the average beach width in this section during the first 25 years of the simulation period. However, during the latter 25 years of the simulation the average shoreline width does diverge slightly between the different Groyne 3 options, however the difference in beach width at the end of the 50-year simulation between the options is less than 1m.

> The erosive trend predicted by the model for all the layouts goes against recent observed trends in this section and so the long-term model projections in this section should be treated with caution. However, the relative change in shoreline positions in this section between the options should be represented by the model.

Section 2

> Section 2 exhibits a very similar trend to Section 1 with no discernible differences between the options.

Section 3

> All of the extension options show a significant improvement over the existing layout in this section. The existing layout exhibits a highly erosive trend. This section of beach is predicted to recede at an average rate of 1m every 3 to 4 years for the existing layout. Conversely, all of the Groyne 3 extension options show either a stable or slightly accreting trend in this section.

> The model indicates that the different length extensions of Groyne 3 will have a small influence on the beach width in Section 3. Compared to the existing length of Groyne 3, the longer Groyne 3 options (20 and 30m extensions) are predicted to result in slightly less beach width (~5m) in this section of beach at the end of the 50-year simulation.

Section 4

> The length of Groyne 3 has a moderate effect on the beach width in Section 4. The existing layout is predicted to show a slight accretion trend for the first 10 years and recede following this, with approximately 5m of recession predicted within the first 25 years of the simulation.

> At the end of the 50-year simulation, the average beach width in this section is predicted to be slightly smaller than the initial width for the existing layout.

> Extending Groyne 3 is predicted to increase the width of the beach. A 20m extension results in a slightly wider beach (~2m wider) compared to the existing layout. Extending the groyne further will result in an additional 10m of beach width compared to the existing layout.

Section 5

> All of the Groyne 3 simulations indicate that Section 5 will accrete by at least 1m every 5 years on average (10m over 50 years).

> Extending Groyne 3 does not have a significant effect on the width of the beach in this section, particularly during the first 25 years of the simulation.

> During the last 25 years of the simulation period, the modelling indicates that the longest Groyne 3 (30m extension) will have a slightly wider beach (~2m wider) compared to the 20m extension.

Section 6

> All of the Groyne 3 simulations indicate that Section 6 will accrete slightly through the 50-year simulation but not as much as for the existing layout. This accretive trend is counter to expectations based on previous experience at other locations and so the long-term model projections in this section should be treated with caution. However, the relative change in shoreline positions in this section between the options should be represented by the model.

> The extended Groyne 3 simulations indicate that the rate of accretion in Section 6 will reduce with the longer groynes. The longest extension is predicted to result in a beach width that is 5m narrower than the beach would be for the existing layout after 50 years.

> The 20m extension provides slightly more beach width (an additional 2-3m) in Section 6 compared to the 30m extension.

3.4.3 Preferred Length of Groyne 3

Based on the modelling results and discussions with the City and DoT, the preferred extension length of Groyne 3 is 20m. This is due to:

> The 20m extension provides similar beach widths in Sections 1, 2 and 3 compared to the 30m extension;

> Extending Groyne 3 an additional 10m (total extension of 30m) is predicted to provide some benefit to Section 4, however, this is at the expense of beach width in Section 6.

> Section 4 (between Groynes 2 and 3) is not a critical area in terms of erosion and a 20m extension of Groyne 2 will stabilise the beach in this section.

> Extending Groyne 3 by 30m is predicted to result in a narrower beach north of Groyne 4, which may increase the likelihood of an additional groyne being required north of Groyne 4 in the future.

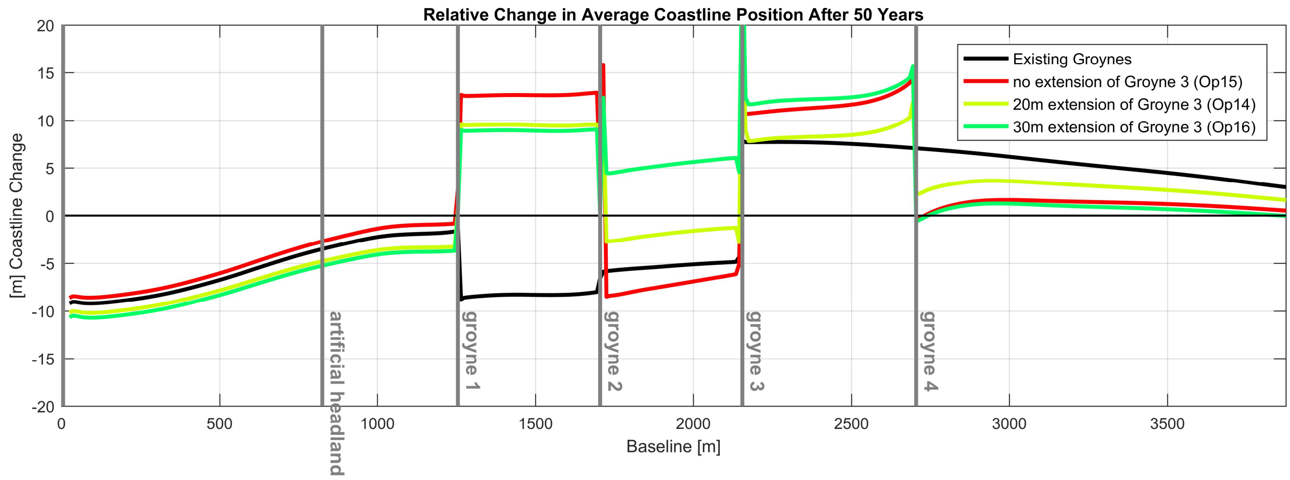

Figure 3-6 1D Plot showing Average Shoreline Position during last Decade of Simulation for Options 14, 15 and 16 relative to that of the Existing Groyne Layout after model has stabilised (years 10 to 20)

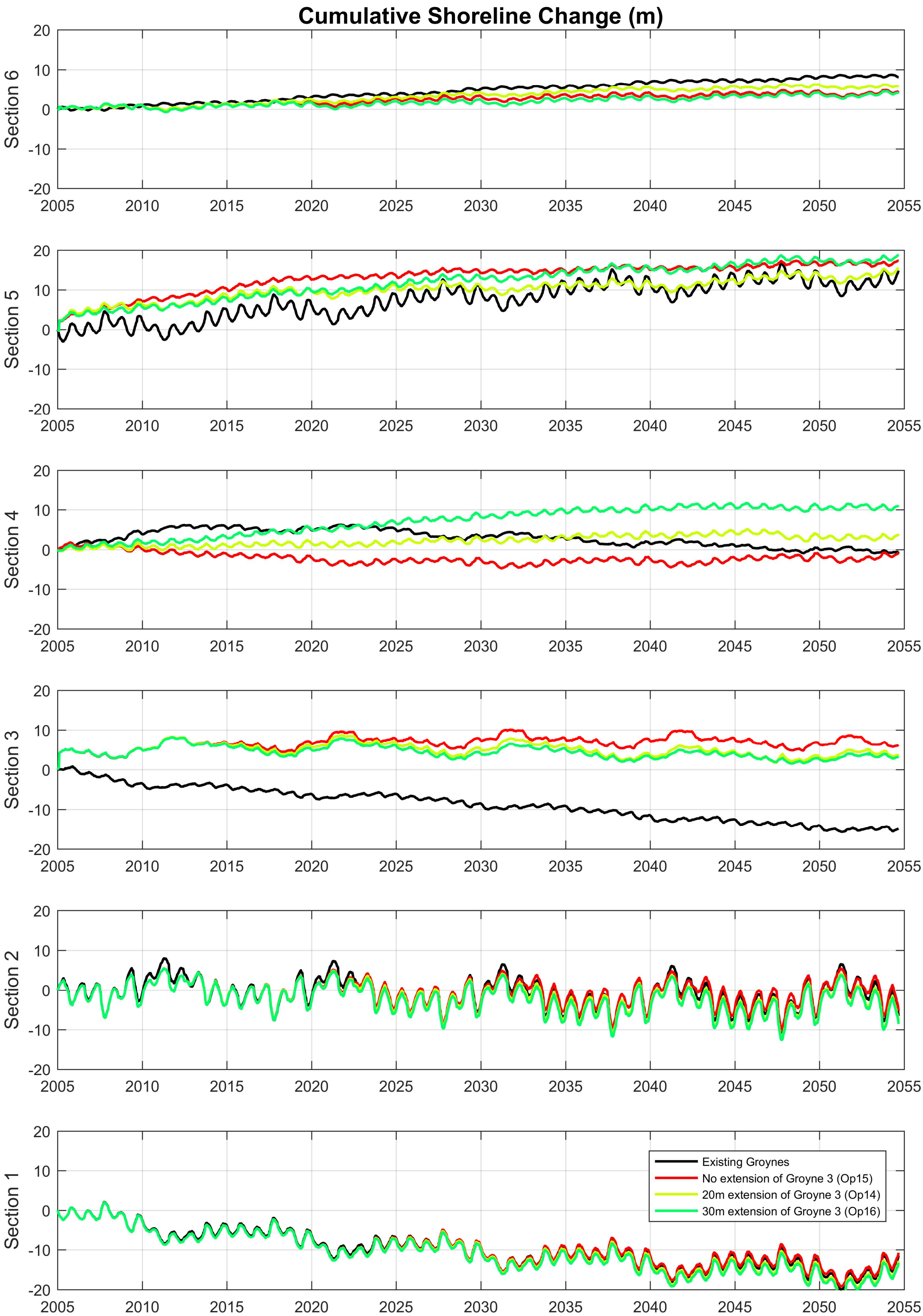

Figure 3-7 Time Series Plots of the Average Beach Width in each Section over the 50-year Simulation for Options 14, 15 and 16

3.5 Groyne 4 Length Assessment

3.5.1 Description of Options

Additional LITPACK simulations were undertaken to optimise the length of Groyne 4. These included:

> **Option 17** – No groyne 4

> **Option 18 –** 50m long Groyne 4 (10m reduction from the Stage 2 length)

> **Option 14 –** 60m long Groyne 4 (Stage 2 length).

> **Option 19 –** 75m long Groyne 4 (15m increase from the Stage 2 length).

These options are presented graphically in Figure 3-8, where the groyne lengths for Options 14, 18 and 19 are shown.

Figure 3-8 Overview of the Groyne 4 Lengths simulated in LITPACK

3.5.2 Results

The LITPACK results are presented in Figure 3-9 and Figure 3-10. These figures show the relative effect of the groynes on the average shoreline position over time. Figure 3-9 shows the relative change in the average beach width along the shoreline, whereas Figure 3-10 presents a time series of the average beach width in each section throughout the 50-year simulation period.

The results are discussed below in terms of each section of Quinns Beach.

Section 1

> The length of Groyne 4 does not appear to have any significant effect on the beach in section 1.

> The erosive trend predicted by the model for all the layouts goes against recent observed trends in this section and so the long-term model projections in this section should be treated with caution. However, the relative change in shoreline positions in this section between the options should be represented by the model.

Section 2

> Section 2 exhibits a very similar trend to Section 1 with no discernible differences between the options.

Section 3

> The length of Groyne 4 does not have a discernible influence on the beach width in Section 3.

Section 4

> The length of Groyne 4 does not have a discernible influence on the beach width in Section 4.

Section 5

> All of the Groyne length 4 simulations indicate that Section 5 will accrete.

> The simulation without Groyne 4 indicates that the beach width in Section 5 will accrete, however the accretion is less than for the existing layout.

> The simulations with Groyne 4 provide additional beach width in Section 5. The 60m Groyne 4 simulation is predicted to provide an additional beach width of 10 to 15m after 50 years.

> Extending Groyne 4 further will provide additional beach width in Section 5, however this is at the expense of beach width north of Groyne 4.

Section 6

> All of the Groyne 4 length simulations indicate that Section 6 will accrete slightly. This accretive trend is counter to expectations based on previous experience at other locations and so the long-term model projections in this section should be treated with caution. However, the relative change in shoreline positions in this section between the options should be represented by the model.

> The simulation with the longest Groyne 4 indicates that the beach north of Groyne 4 would accrete at a slower rate compared to the existing groyne field and the shorter Groyne 4 lengths.

3.5.3 Preferred Length of Groyne 4

Based on the modelling results and discussions with the City and DoT, the preferred length of Groyne 4 is 60m. This is due to:

> The 60m long Groyne 4 provides similar beach widths in sections 1, 2, 3 and 4 compared to the longer 75m groyne.

> Extending Groyne 4 an additional 15m (total length of 75m) is predicted to provide some additional benefit to Section 5, however this is at the expense of beach width in Section 6. The model predicts that the shorter Groyne 4 already provides sufficient additional beach width in Section 6, and the benefit offered by a longer groyne is not substantial.

> The longer Groyne 4 is predicted to result in a narrower beach north of Groyne 4, which may increase the likelihood of an additional groyne being required north of Groyne 4 in the future.

> The simulations without Groyne 4 provides less beach width in Section 5 compared to the existing layout.

> The beach width north of Groyne 4 is similar for the simulations with a 60m Groyne 4 and no Groyne 4. This indicates that the 60m Groyne 4 is short enough to allow sufficient sand to bypasses to feed the beach to the north.

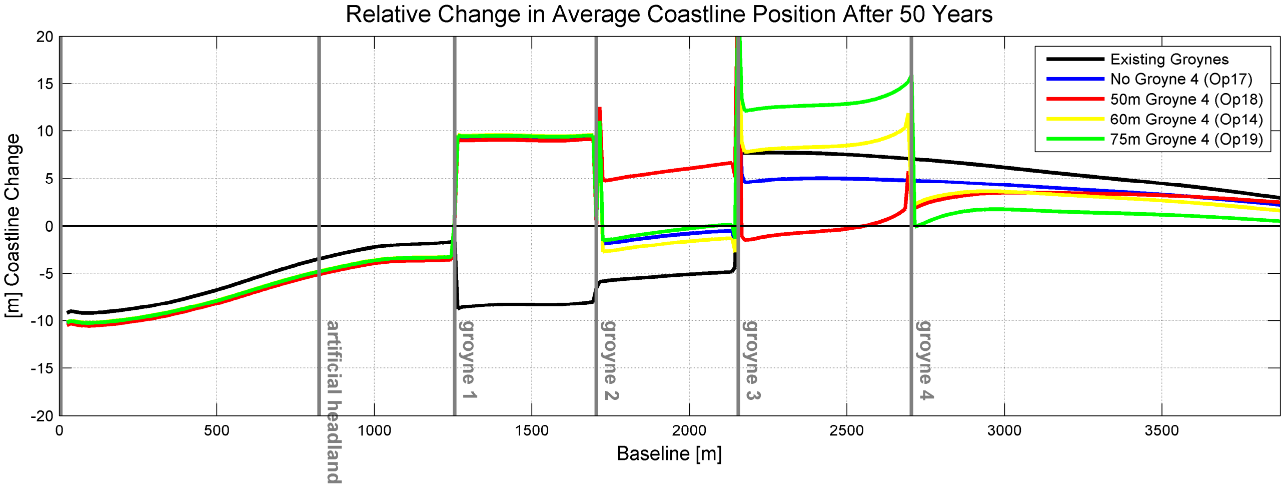

Figure 3-9 1D Plot showing Average Shoreline Position during Last Decade of Simulation for Options 14 and 17-19 relative to that of the Existing Groyne Layout after Model has stabilised (years 10 to 20)

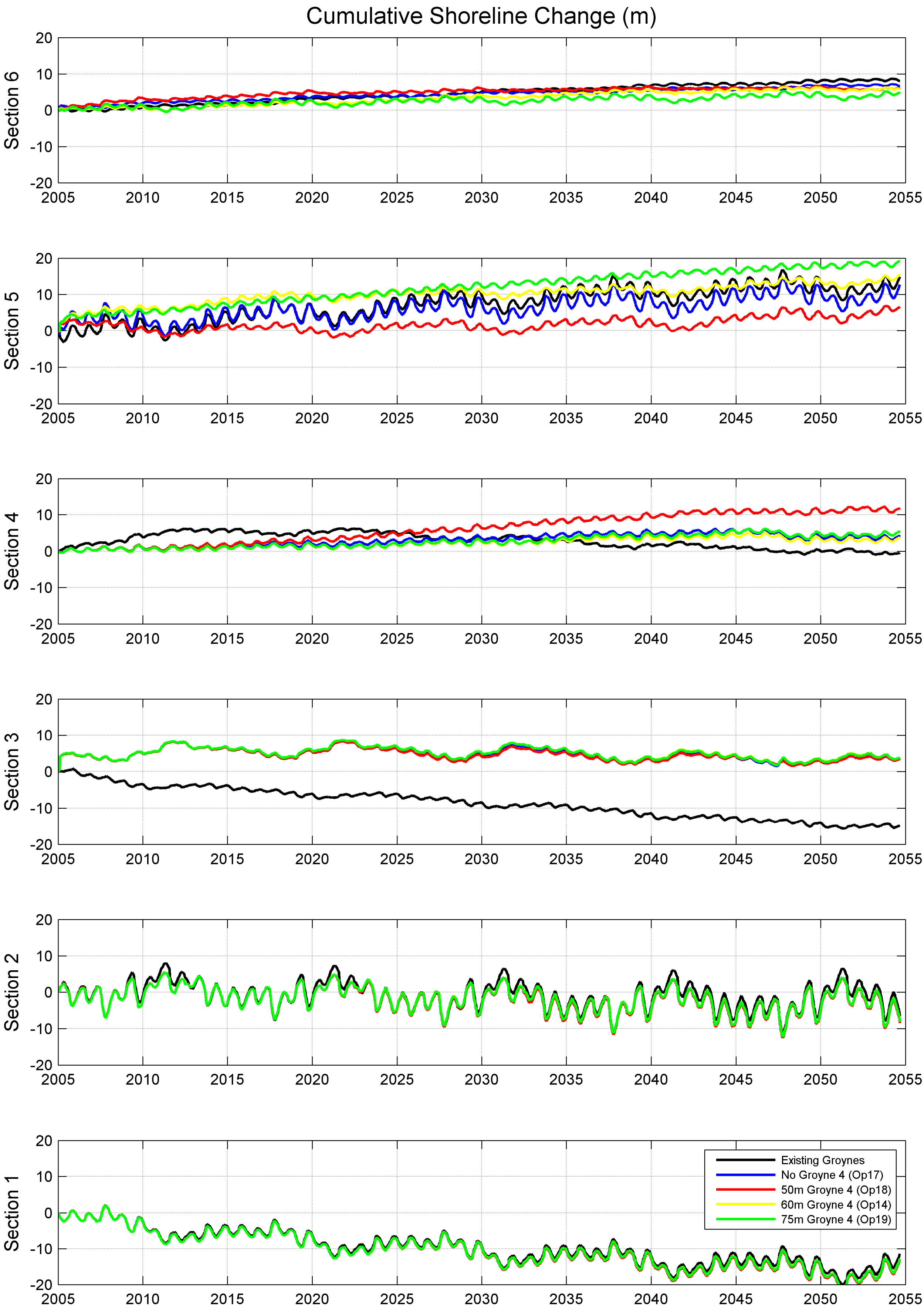

Figure 3-10 Time Series Plots of the Average Beach Width in each Section over the 50-year Simulation for Options 14, 17, 18 and 19

3.6 Groyne 4 Location Assessment

3.6.1 Description of Options

An assessment of the optimum location of Groyne 4 was undertaken using LITPACK. This included locating Groyne 4 100m north and south of its location in Stage 2 of the project. The scenarios modelled were:

> **Option 14** – Stage 2 Groyne 4 location

> **Option 20 –** Groyne 4 moved 100m to the south from the stage 2 location

> **Option 21 –** Groyne 4 moved 100m to the north from the stage 2 location

In each case the length of groyne seaward of the 0m AHD contour was kept the same at 20m. The Groyne 4 locations simulated are shown in Figure 3-11.

Figure 3-11 Overview of the Groyne 4 Locations simulated in LITPACK

3.6.2 Results

The predicted shoreline changes due to locating Groyne 4 up or down the coast are presented in Figure 3-12 and Figure 3-13. These figures show that the majority of the shoreline differences are located between Groynes 2 and 4. No differences are predicted south of Groyne 2.

The model simulations predict that if Groyne 4 were to be constructed 100m to the south, the beach width between Groynes 3 and 4 would not accrete as much as the simulations with the Option 12 location. Similarly, the simulations indicate that by moving Groyne 4 further north the beach width between Groynes 3 and 4 would be smaller than the other two options.

These LITPACK results are counter-intuitive, as a consistent trend between the options would be expected. However, upon further examination of the Groyne 4 locations, the beach naturally changes orientation just to the north of Groyne 4 between the Option 12 location and the location that was modelled 100m to the north. This reduces the effective length of the northern Groyne 4 as the head of the groyne is slightly further east compared to the other Groyne 4 locations.

3.6.3 Preferred Location of Groyne 4

Based on the modelling results the same location of Groyne 4 as was included in the Stage 2 layout is proposed. This is due to:

> This location allows for the easiest access to the back of the groyne compared to the other two options,

> To achieve the same level of benefit between Groynes 2 and 3 the northern Groyne 4 location would require a much larger Groyne 4,

> Moving Groyne 4 to the south is predicted to result in less beach width between Groynes 3 and 4 compared to the middle location.

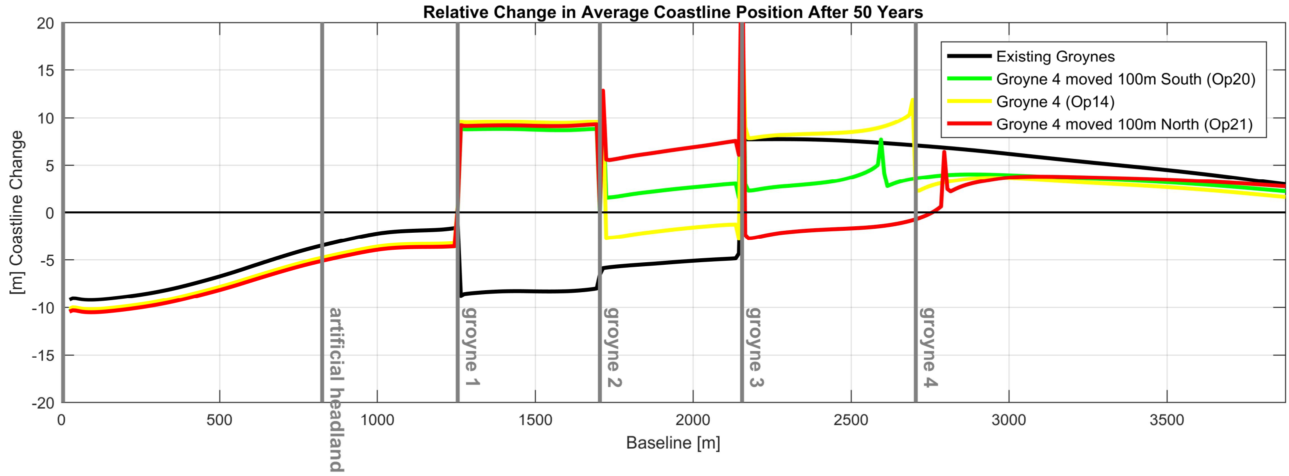

Figure 3-12 1D Plot Showing Average Shoreline Position during Last Decade of Simulation for Options 14, 20 and 21 relative to that of the Existing Groyne Layout after Model has Stabilised (years 10 to 20)

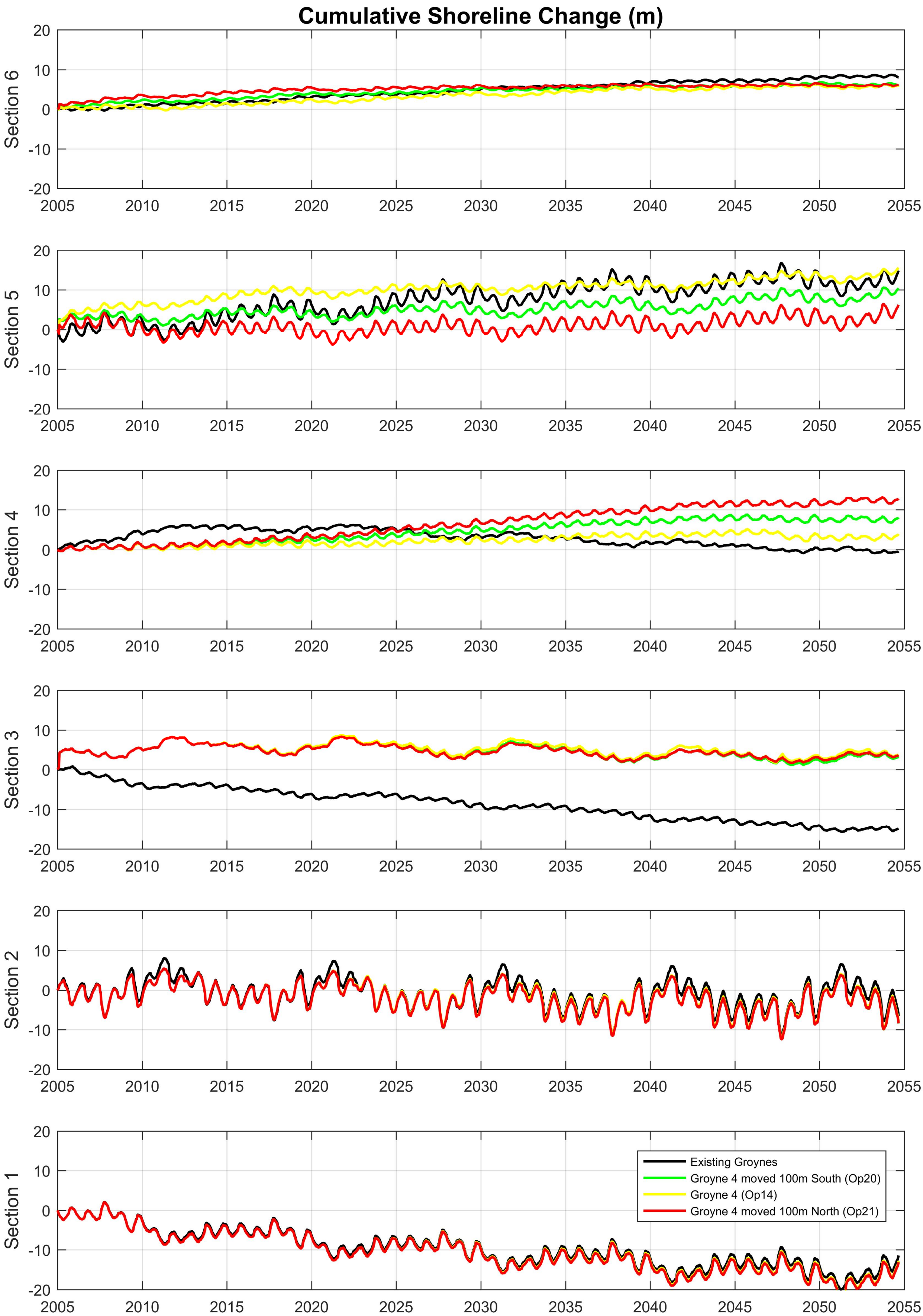

Figure 3-13 Time Series Plots of the Average Beach Width in each Section over the 50-year Simulation for Options 14, 20 and 21

3.7 Refined Layout Recommendation

Based on the 50-year LITPACK simulations carried out during this stage of the project, it is recommended that the refined layout consist of the following:

1. Groyne 2 be extended by 45m. This extension is 15m less than the extension proposed in Stage 2,
2. Groyne 3 be extended by 20m. This is the same as the extension proposed in Stage 2,
3. A Groyne 4 be constructed at the same location as was proposed during Stage 2, having the same length of 60m as was proposed in Stage 2.

Of the options assessed in this stage, the above combination is predicted to:

1. Have little impact compared to the existing layout south of Groyne 1,
2. Provide a substantially wider beach between Groynes 1 and 2,
3. Stabilise the beach between Groynes 2 and 3,
4. Provide a wider, and less variable beach width between Groynes 3 and 4, and
5. Limit impacts to beach width to the north of the proposed Groyne 4.

4 Storm Erosion Assessment

4.1 Introduction

The storm erosion assessment utilised the calibrated XBEACH storm erosion model detailed in Cardno (2015) and subsequently applied during the options assessment during Stage 2 of the project in Cardno (2016). As part of Stage 3 of the project, the same simulation scenario that was applied to the different options assessed in Cardno (2016) was applied to the refined layout (Option 14) developed through the longshore sediment transport modelling detailed in Section 3 of this report. This allowed comparison of the performance of the refined layout (Option 14) to that of both the existing layout and the layout chosen from Stage 2 of the project (Referred to as “MCA2/LITPACK Option 12” in Cardno (2016)) under extreme storm conditions.

As per Cardno (2015) and Cardno (2016), the model was calibrated using measured data for a one week simulation covering a storm in September 2014, when pre- and post-storm beach surveys were available. The simulation period was for six weeks from 19th August 2013 to 30th September 2013, which included three storms that significantly impacted Quinns Beach, and at the time resulted in significant erosion in Section 2 adjacent to Frederick Stubbs Park.

As per the XBEACH modelling undertaken during Stage 2 (Cardno 2016), no renourishment was included in the initial XBEACH model bathymetry. The bathymetry utilised in the models is the pre-September 2014 model calibration bathymetry.

The limitations and uncertainties of the storm erosion modelling are discussed in detail in section 7.

4.2 Simulation Results

The XBEACH model results provide detailed spatial information on the cross shore and longshore movement of sediment in the model domain under storm conditions. This section outlines the model results for the refined layout and compares them to that of the existing layout and the layout chosen from Stage 2 of the project.

Similar to the LITPACK analysis, a time series plot showing the cumulative change in the average width of each section is presented in Figure 4-1 (positive values represent a cumulative increase in beach width (accretion) in the given beach section and negative values represent a cumulative decrease in beach width (erosion) in the given beach section. This first subplot shows the chosen time delineation between the three storm events (grey vertical lines), which are also shown in Table 4-1. The measured conditions at the Rottnest Wave Rider Buoy during the simulation period are presented in Figure 4-2. It should be noted that there can be significant sediment transport without a significant change in the location of the 0m AHD contour, as can be observed in some of the spatial outputs. For example, there might be significant erosion of the beach berm and dune above the 0m AHD contour.

The majority of model result plots are presented in Appendix E. The first set of figures shows total accretion and erosion per section over the entire model period time frame to allow for better comparison between each layout. The final set of figures in Appendix E shows the locations of 19 cross shore transects along the study area, followed by a plot of the initial and final beach profiles along each transect for each option. The transects are numbered from 1 in the south of Section 1 of the project area to 19 at the northern end of Section 6 of the project area.

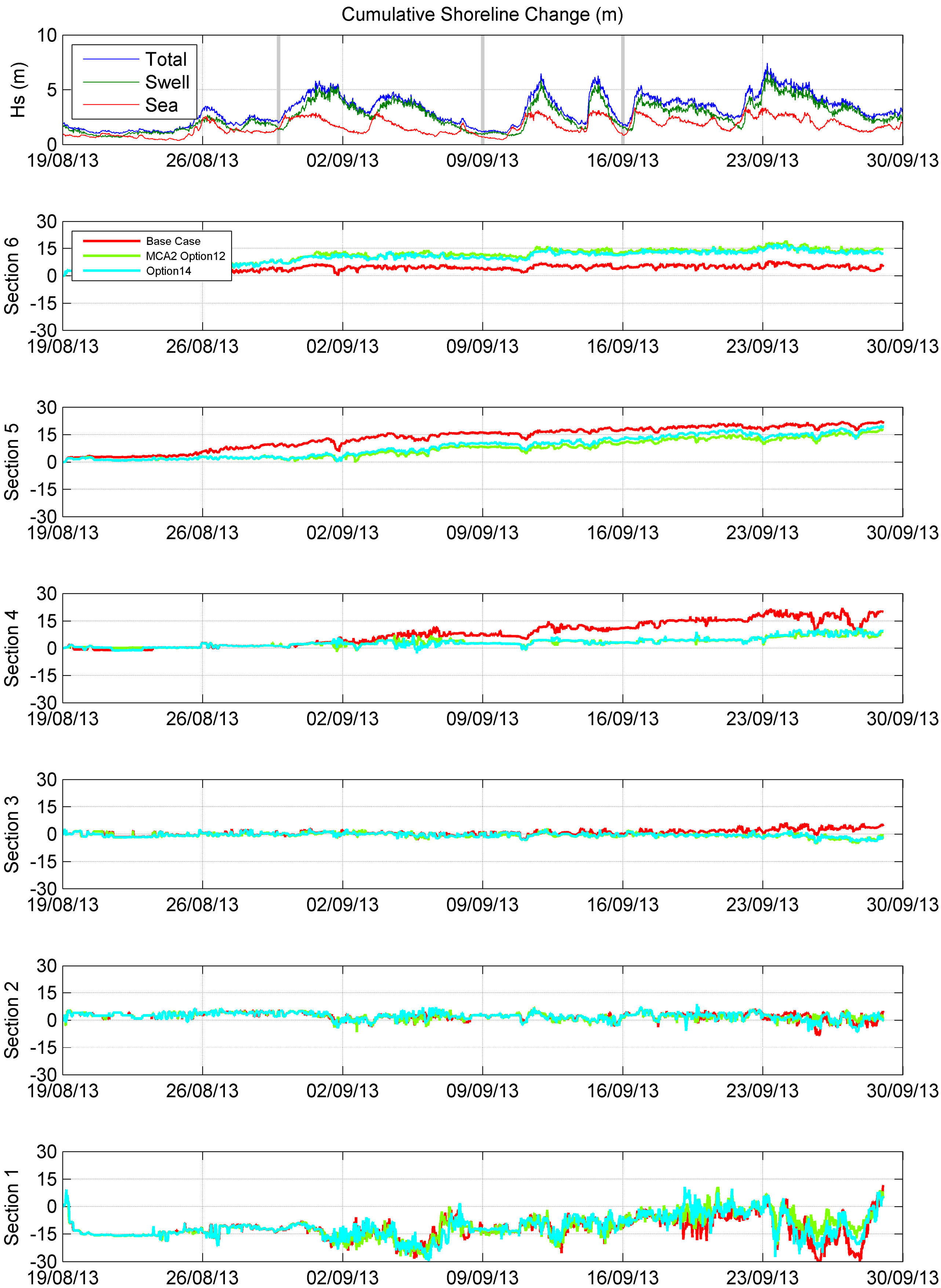

Figure 4-1 Cumulative Movement of the Average Beach Width Across each Section in Metres

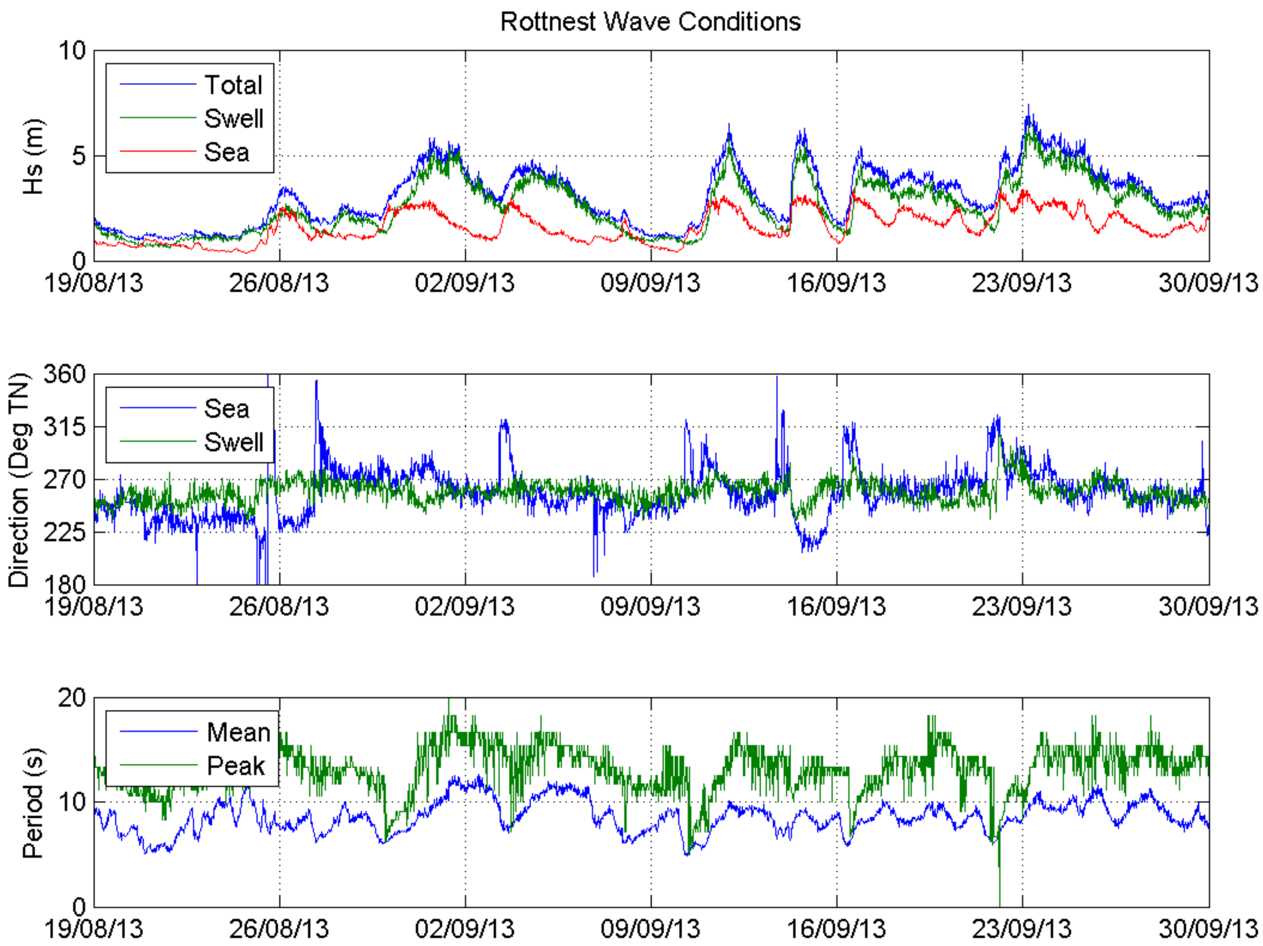

Figure 4-2 Rottnest Island Wave Buoy Measured Conditions for the Duration of the XBEACH Model Simulations

Table 4-1 XBEACH Simulation Important Start Times

Simulation warm up	Storm 1	Storm 2	Storm 3	Simulation end
19/08/13	29/08/13 19:00	09/09/13 00:00	16/09/13 00:00	03/09/13 00:00

In general, as the refined layout is basically the chosen layout from Stage 2 of the project with a small adjustment to only one of the groynes, the results of the XBEACH simulation for the refined layout (Option 14) are very similar to that of the chosen layout from Stage 2 of the project (MCA2/Option 12). For convenience, the trends observed over the simulation period in Cardno (2016) are repeated here as they are also applicable to the additional XBEACH results for the refined layout (Option 14):

> A significant flux of sediment from north to south, with sections experiencing a significant shoreline rotation and 'filling' of each structure on its northern side, and erosion immediately to the south.

> A smaller cross-shore flux of sediment in the offshore direction, with significant erosion occurring above the initial 0 m AHD contour in most sections, and almost no accretion occurring above the same line.

> There is virtually no significant change in the bed level outside of the range ±3 m AHD.

> The significant southward longshore sediment flux is also observed during the simulation warm up period, although significant cross shore transport was not readily observable from the spatial output maps in Appendix D. This period did experience a relatively small (~2.5 m) increase in wave height off Rottnest which included two spikes in the sea direction from the north, as shown in Figure 4-2.

> The second storm had a relatively minor impact on the study area both in terms of bed level change, with some minor recovery observed in generally eroding areas. This is consistent with anecdotal evidence reported at the time.

> All options behaved relatively the same in Sections 1 and 6 in terms of bed level change over the entire simulation. This included significant deposition immediately south of the artificial headland.

> Section 4 behaved similarly for all three options, experiencing mild accretion in the southern half and mild erosion in the northern half. The base case experienced section wide accretion here due to the lack of impediments to sediment flow from the north that were present in the three conceptual options.

> The southern end of Section 5 accretes significantly for all options and the base case.

The following observations were made in Cardno (2016) specifically for the MCA2/Option 12 layout and are repeated here for convenience as they are also applicable to the XBEACH results for the refined layout (Option 14).

In general this option performs adequately throughout the study area: it protects the beach in front of the car park and the dog beach without putting as much pressure on erosion pinch points. A summary of observations from the analysis figures is as follows:

> Average beach width by section is relatively stable for Sections 1 to 3, while increases of 5 to 15m are observed in Sections 4 to 6.

> Within Section 2, the 0m AHD contour remains within around 5 m of its starting position. However significant erosion occurs along the whole section both above and below the 0m AHD contour.

> The extension of Groyne 2 results in significantly more erosion to the northern half of Section 3 compared with the existing layout. This erosion occurs at all phases of the simulation including, to a lesser extent, Storm 2. This suggests that sediment bypasses Groyne 2 moving from north to south under the existing layout but the extended Groyne 2 blocks this sediment movement under Option 14. It should be noted that the XBEACH simulations do not include the capital renourishment which is part of the refined layout. The capital renourishment is designed to increase the beach buffer in this section of Quinns Beach which will counteract the predicted storm erosion in the northern half of Section 3.

> The addition of Groyne 4 reduces erosion to its north while only producing a very minor amount of erosion to its south.

5 Extreme Design Criteria

Simulation of an extreme wave and water level event was undertaken in order to facilitate structural design of the groynes.

5.1 Design Life

The City of Wanneroo specified that the design life of all the groyne structures was to be 50 years.

5.2 Design Criteria

To decrease the likelihood of severe damage during the proposed design life, 100-year Average Recurrence Interval (ARI) wave and water level design criteria have been utilised to determine each structures' required characteristics. A 100-year ARI event has approximately a 40% chance of being reached or exceeded within a 50-year design life.

The nearshore wave conditions will be depth limited, meaning that the design wave height is dependent on the offshore wave conditions, the water level (tide, surge, sea level rise and wind and wave setup) and the depth.

For groyne armour design the combination of 100-year wave height and 100-year water level (plus sea level rise) was applied in the design calculations, meaning that the armour was designed to a combined ARI event of greater than 100 years. This slightly conservative approach was taken as combinations of the 100-year wave height and 25-year water level, and vice versa, resulted in wave heights at the groynes only a few centimetres lower than for the 100-year wave height and 100-year water level combination.

For overtopping, criteria were assessed against 1-year and 100-year combinations of wave height and water level plus sea level rise. Based on Cardno Lawson Treloar (2010), the combination of the 100-year ARI wave height and the 100-year ARI water level would have a combined ARI significantly greater than 100 years. For the 1-year event the combination of the 1-year ARI wave height and the 1-year ARI water level have been considered. For the 100-year ARI, combinations of events have been simulated assuming the 100-year ARI wave height occurs at the same time as a 25-year ARI water level (still greater than 100-year ARI, but understanding that there is some dependence between peak wave and elevated water level conditions), and alternatively that a 100-year ARI water level occurs at the same time as a 25-year ARI wave height.

5.2.1 Waves

Design wave heights for each groyne were determined by simulating the 100-year ARI wave condition based on analysis of the Rottnest Wave Buoy (48m water depth) installed and maintained by DoT (Li et al 2012) using the SWAN wave model developed and applied in previous stages of the project (Cardno 2015, 2016). As per the results of the wave model calibration detailed in Appendix C, the variable roughness map was applied in the Extreme SWAN simulation. The wave condition applied at the boundary of the SWAN model is detailed in Table 5-1. The significant wave heights in Table 5-1 were obtained from Li et al. (2012) while the associated wave periods and peak wave direction were determined by analysis of the Rottnest Wave Buoy record.

Table 5-1 Wave Conditions applied at Offshore Boundary of Extreme SWAN Model Simulation

ARI (years)	Significant Wave Height (m)	Peak Wave Period (sec)	Peak Wave Direction (ºN)
1	6.91	12	270
5	7.94	13	270
25	8.83	13	270
100	9.53	13	270

Design wave heights were extracted from the SWAN model results 25m offshore of the head of each groyne (approximately ½ a wavelength), and at 5m intervals along each of the groyne lengths. Outputs included significant wave height, mean wave period, peak wave period, wave direction and wave setup.

5.2.2 Water Levels

The extreme water levels determined in Cardno (2016) from a Weibull extreme value analysis of tidal gauge records from Two Rocks and Hillarys were applied in the groyne design and are presented in Table 5-2.

Table 5-2 ARI Extreme Water Levels

ARI	100-year ARI Extreme Water Level (m AHD)
1	+0.96
5	+1.12
25	+1.21
100	+1.34

5.2.3 Allowance for Sea Level Rise

Design of coastal structures must allow for future projected sea level rise. The allowance made for future sea level rise was based on the recommendations published in DoT (2010). The recommended allowance for sea level rise in DoT (2010) is shown in Figure 5-1. Based on this predicted sea level rise curve, the sea level rise between 2015 and 2065 was estimated and is presented in Table 5-3. This allowance for sea level rise was applied in the structural design of the groyne structures.

Table 5-3 Allowance for Sea Level Rise from 2015 to 2065 estimated from DoT (2010)

Allowance for Sea Level Rise over 50-year Design Life (m)
0.34

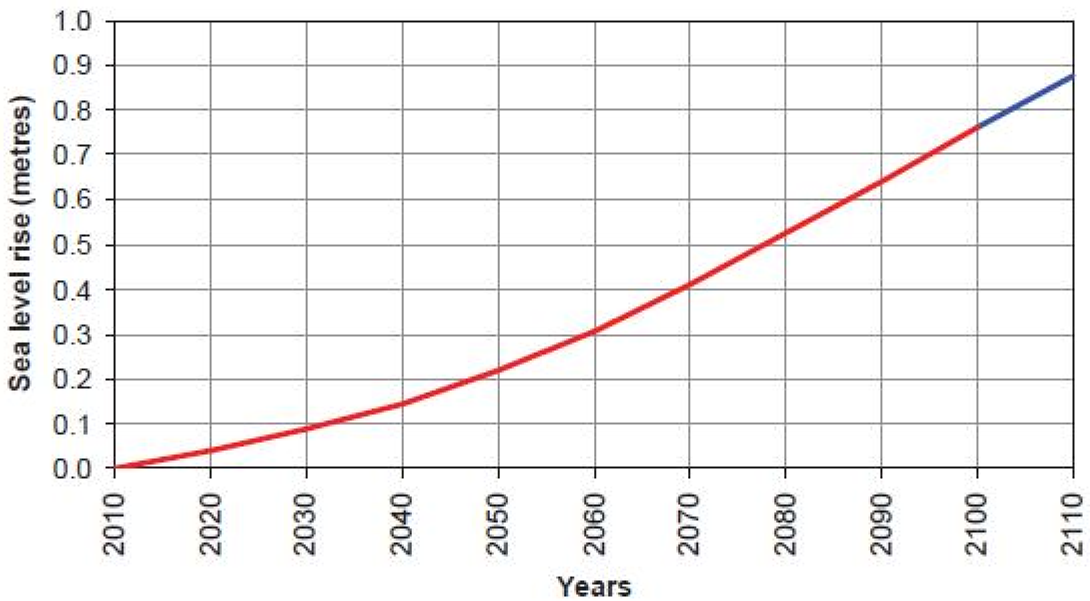

Figure 5-1 Sea Level Rise in Western Australia (DoT 2010)

5.2.4 Damage Criteria

The groynes were designed to a damage criterion of 0-5% displacement of armour units during the design event. This level of damage is readily repairable.

6 Detailed Design

This section details the design process and considerations for each element of the scope of work.

6.1 Groyne 2

It is understood that the existing groynes have suffered regular, ongoing damage and require routine maintenance to replace armour rocks and repair the crest of the structures. This damage is understood to be due to the existing groynes having a relatively low crest level at the head, as well as the structure not including a filter layer between the armour rock and the core. The lack of a filter (or secondary armour layer) will allow the smaller material from the core to move out of the groyne through the gaps in the primary armour layer during storm events.

6.1.1 Rock Armour

Both the extended and new groyne structures will consist of core, secondary and primary armour.

The mean armour mass has been calculated using the Van Der Meer formula with the following inputs:

> Minimum armour density = 2000 kg/m^3 (limestone)

> Storm duration = 6 hours

> Structure porosity = 0.4 (cross section consisting of 2 layers of armour, 2 layers of secondary armour and a well graded permeable core). Note, this assumes there is no geotextile on the slope of the structure.

> Wave conditions from the outputs of the SWAN modelling.

Table 6-1 summarises the minimum required mean armour rock mass required on Groyne 2. The calculations have been carried out for three slopes (1V:1.5H, 1V:2H and 1V:2.5H) as well as two separate damage criteria (S=2 and 4).

During the initial design, typical cross sections were developed to compare the required armour, underlayer and core volumes for the different slope options. Based on this assessment it was found that the cross sections with the flatter slopes would require larger volumes of core, underlayer and armour rock compared to a comparable cross section with 1V:1.5H slope. The greater volume of material would equate to a more expensive structure.

Designing for a damage level of S=2 is also recommended as this will minimise the maintenance requirements of the groyne.

The existing groynes are armoured with 1T, 3T and 5.5T armour rocks. The intention is to re-use this armour where possible, and so the extended groynes have been designed with four classes of armour rock, which are 1T, 3T, 5.5T and 6.6T.

As a check, the primary armour size was calculated using Hudson's formula. The results show Hudson's formula indicate much larger primary armour would be required on Groyne 2. Hudson's formulae does not explicitly include wave period in the calculation. Due to the relatively short wave periods at the site, Cardno believes that the Van Der Meer formulae gives more accurate estimates of the required primary armour size. It should be noted that the results using Hudson's formulae are very sensitive to rock density, with a density of 2,100 kg/m3 reduces the required primary armour mass by 20% and a density of 2,200 kg/m3 reduces the required primary armour mass by 35%. Both these densities are not uncommon for quarried limestone.

Table 6-1 Groyne 2 Armour Mass

		SWAN RESULTS					Armour Mass, M50 (kg)						
Distance from head	Bed Level	100yr Wave + 100yr WL + 50yr SLR					S=2 (Start of Damage)			S=4 (repairable)			Hudson
Towards shore (m)	m AHD	Hs (m)	H1/10	Hb	Breaking (H1/10>Hb)	Tm01 (sec)	1V:1.5H	1V:2H	1V:2.5H	1V:1.5H	1V:2H	1V:2.5H	1:1.5H
25m offshore	-4.8	2.31	2.93	5.09	No	4.9	6,616	4,293	3,042	4,293	2,810	2,000	9,740
0	-3.9	2.24	2.84	4.35	No	5.0	6,224	4,001	2,886	4,097	2,662	1,882	8,870
5	-3.6	2.20	2.79	4.12	No	5.0	5,972	3,906	2,735	3,906	2,590	1,825	8,410
10	-3.3	2.16	2.74	3.89	No	5.0	5,727	3,722	2,662	3,813	2,450	1,769	7,970
15	-3.0	2.12	2.69	3.65	No	5.0	5,488	3,543	2,590	3,632	2,382	1,715	7,540
20	-2.7	2.08	2.64	3.42	No	5.0	5,256	3,456	2,450	3,456	2,250	1,609	7,130
25	-2.4	2.03	2.58	3.18	No	5.0	5,031	3,286	2,382	3,370	2,185	1,557	6,650
30	-2.1	1.99	2.52	2.93	No	5.1	4,812	3,122	2,250	3,203	2,061	1,458	6,200
35	-1.8	1.94	2.46	2.69	No	5.1	4,600	2,963	2,122	3,042	1,941	1,410	5,770

1 T	3 T	5.5 T	6.6 T

The secondary armour layers should be 1/10th the mass of the primary armour and are presented in Table 6-2.

Table 6-2 Secondary Armour Mass

Primary Armour M_{50} Mass	Secondary Armour M_{50} Mass
6.6T	0.7T
5.5T	0.55T
3T	0.3T
1T	No secondary armour layer

6.1.2 Crest Level

The crest levels of the groynes have been set according to the allowable overtopping rates as per the criteria presented in Table 6-3 below.

Table 6-3 Wave Overtopping Criteria Specified for Design (EurOtop 2016)

ARI event (years)	Criteria	Overtopping discharge limit (l/s/m)
1	Pedestrian Safety	0.3 l/s/m
100	Crest stability	5 l/s/m

Based on the SWAN modelling results, wave overtopping discharges were estimated for each combination of wave and water level conditions using the Deltares Neural Network Overtopping Calculator (Van Gent et al 2007). As described in section 5.2, the combination of 1-year ARI wave height and 1-year ARI water level was applied for the 1-year ARI overtopping criteria and the combination of 100-year ARI wave height and 25-year ARI water level, and vice versa, was applied for the 100-year ARI overtopping criteria. Sea level rise at the end of the 50-year design life and wave setup were also included in the overtopping calculations.

For each section of groyne, wave overtopping rates were estimated for crest levels of 4, 4.5 and 5m AHD. Waves were assumed to approach perpendicular to the head, and at a 45 degrees oblique angle on the trunk.

The results indicate that the criterion that governs the crest level is the criterion for pedestrian safety. To achieve overtopping rates less than 0.3 l/s/m in a 1-year ARI event, the minimum crest levels are required to be:

Head of groyne: +5.0m AHD

Trunk of groyne: +4.5m AHD

The crest level of the groyne can be lowered as the waves become smaller towards shore. The design has included the crest along the new trunk section at +4.5m AHD, which then slopes down to +2.5m AHD at the landward (root) of the groyne. This lower crest level at the root of the groyne allows easier pedestrian access across the trunk of the groyne at the back of the beach.

For ease of construction, the head of the groyne has a crest level of +6.0 m AHD, which is one additional layer of armour rock.

6.1.3 Toe Detail

Given the shallow nature of the site, the toe of the groyne has been designed as an extension of the primary armour layer.

The geophysical investigation indicates that there is generally around 2-3m of sand at the site, limiting potential scour. As the scour potential in deeper water is low, for ease of construction the toe has been designed to sit on the existing seabed where the existing seabed is lower than -1m AHD.

Where the seabed is shallower than -1m AHD the toe of the groyne has been designed as an embedded toe to allow for potential scour.

As there is no core or underlayer under the toe, a geotextile filter (Texcel 600R or similar) is proposed to be placed under the bedding layer to stop piping of sand through the toe and the toe from sinking. Care will need to be taken so as not to damage the geotextile filter during construction.

6.1.4 Tie-in with Existing Groyne

The design drawing of the existing groyne indicates that the crest level for the existing groynes is +4m AHD at the head, and +2m AHD along the trunk. The crest level of the groynes taken from a survey carried out in 2008 by McMullen Nolan was a maximum of 3.98m AHD, which matches the design drawing.

The design drawing provides an indication of the core of the existing groynes. The new core should be built to cover the core of the existing groyne on all sides to limit any future seepage of core material. There should be a minimum cover of one layer of the new core (37cm) on top of the existing core on the slopes of the groyne structure and the existing groyne core will need to be removed to facilitate this if necessary.

6.2 Groyne 3

Groyne 3 has been designed in a similar manner to Groyne 2, with the exception of the root (landward end of the groyne). A concrete access path currently joins onto the existing root of the groyne. The design has minimised disturbance in this area in order to avoid disrupting the existing access road.

6.2.1 Rock Armour

Table 6-3 summarises the minimum required mean armour rock mass required on Groyne 3. The calculations have been carried out for three slopes (1V:1.5H, 1V:2H and 1V:2.5H) as well as two separate damage criteria (S=2 and 4).

Similar to the analysis carried out on Groyne 2, typical cross sections were developed to compare the required armour, underlayer and core volumes for the different slope options. Based on this assessment it was found that the cross sections with the flatter slopes would require larger volumes of core, underlayer and armour rock compared to a comparable cross section with 1V:1.5H slopes. The greater volume of material would equate to a more expensive structure.

Designing for a damage level of S=2 is also recommended as this will minimise the maintenance requirements of the groyne.

The existing groynes are armoured with 1T, 3T and 5.5T armour rocks. The intention is to re-use this armour where possible, and so the extended groynes have been designed with four classes of armour rock which are 1T, 3T, 5.5T and 6.6T.

As a check, the primary armour size was calculated using Hudson's formula. The results show Hudson's formula indicate larger primary armour would be required on the head of Groyne 3. Hudson's formulae does not explicitly include wave period in the calculation. Due to the relatively short wave periods at the site, Cardno believes that the Van Der Meer formulae gives more accurate estimates of the required primary armour size. It should be noted that the results using Hudson's formulae are very sensitive to rock density, with a density of 2,100 kg/m3 reduces the required primary armour mass by 20% and a density of 2,200 kg/m3 reduces the required primary armour mass by 35%. Both these densities are not uncommon for quarried limestone.

Table 6-4 Groyne 3 Armour Mass

		SWAN RESULTS					Armour Mass, M50 (kg)						
	Bed Level	100yr Wave + 100yr WL + 50yr SLR					S=2 (Start of Damage)			S=4 (repairable)			Hudson
Distance from head	m AHD	Hs	H1/10	Hb	Breaking (H1/10>Hb)	Tm01	1V:1.5H	1V:2H	1V:2.5H	1V:1.5H	1V:2H	1V:2.5H	1:1.5H
25m offshore	-3.1	2.13	2.71	3.76	No	5.1	6,224	4,097	2,886	4,097	2,662	1,941	7,710
0	-1.7	1.86	2.36	2.66	No	5.2	4,705	3,042	2,185	3,122	2,000	1,410	5,090
5	-1.3	1.72	2.18	2.29	No	5.2	3,906	2,519	1,825	2,590	1,661	1,185	4,010
10	-0.8	1.56	1.99	1.92	Yes	5.2	3,203	2,061	1,458	2,122	1,363	986	5,310
15	-0.3	1.39	1.77	1.55	Yes	5.2	2,450	1,609	1,144	1,609	1,063	746	2,880
20	0.2	1.19	1.51	1.17	Yes	5.3	1,769	1,144	810	1,144	746	549	1,240
25	0.7	0.95	1.21	0.80	Yes	5.4	986	716	500	657	454	333	400
30	1.2	0.61	0.78	0.41	Yes	5.7	182	170	159	119	110	101	50

1 T	3 T	5.5 T	6.6 T

The secondary armour layers should be 1/10th the mass of the primary armour and are presented in Table 6-5.

Table 6-5 Secondary Armour Mass

Primary Armour M_{50} Mass	Secondary Armour M_{50} Mass
6.6T	0.7T
5.5T	0.55T
3T	0.3T
1T	No secondary armour layer

6.2.2 Crest Level

The crest level of the groyne has been set according to the allowable overtopping rates as per the criteria presented in Table 6-3.

Based on the SWAN modelling results wave overtopping discharges were estimated for each combination of wave and water level conditions using the Deltares Neural Network Overtopping Calculator (Van Gent et al 2007).

For each section of groyne, wave overtopping rates were estimated for crest levels of 4, 4.5 and 5m AHD. Waves were assumed to approach perpendicular to the head, and at a 45 degree oblique angle on the trunk.

The results indicate that the criteria that governs the crest level is the criteria for pedestrian safety. To achieve overtopping rates less than 0.3 l/s/m in a 1 year ARI event the minimum crest levels are required:

> **Head of groyne: +5.0m AHD**
> **Trunk of groyne: +4.5m AHD**

The crest level of the groyne can be lowered as the waves get smaller towards shore. The design has included the new trunk section at +4.5m AHD, and then slopes down to +2m AHD where it joins the existing root of the groyne to match the existing concrete access path. The southern side of Groyne 3 was modified when the concrete access path was constructed and some scour protection was added to protect the end section of the concrete access path. To avoid disturbing the existing concrete access path to Groyne 3, the extension of Groyne 3 includes modifications on the north side of the groyne for the landward 15m of the groyne. The southern side of this section of Groyne 3 should be tied into the existing scour protection and concrete access path. This tie-in will be verified by the contractor on site once the exact location and condition of the scour protection is verified. The crest level of the groyne has been designed to match the existing access path level at the root of the groyne where the existing concrete access path connects to the existing groyne.

For ease of construction the head of the groyne has a crest level of +6.0 m AHD, which is one additional layer of armour rock rather than stepping the core to construct a smaller crest at the head.

6.2.3 Toe Detail

A similar toe detail for Groyne 3 has been designed as for Groyne 2. Where the bed level is below -1m AHD the toe has been designed to sit on the existing seabed, and has been designed as an embedded toe where the seabed is shallower than -1m AHD.

As there is no core or underlayer under the toe a geotextile filter is proposed to be placed under the bedding layer to stop piping of sand through the toe.

6.2.4 Tie-in with Existing Groyne

The drawings provide an indication of the core of the existing groynes. The new core should be built to cover most of the core of the existing groyne on all sides to limit any future seepage of core material. The landward approximately 15m of the existing Groyne 3 is not proposed to be altered to avoid any changes to the existing concrete access path. There should be a minimum cover of one layer of the new core (37cm) on top of the existing core on the slopes of the groyne structure and the existing groyne core will need to be removed to facilitate this if necessary.

6.3 Groyne 4

Groyne 4 has been designed in a similar manner to Groynes 2 and 3, with the exception of the root. A concrete access path has been designed to tie into the root of the new Groyne 4 and will be constructed as part of the works.

6.3.1 Rock Armour

Table 6-6 summarises the minimum required mean armour rock mass required on Groyne 4. The calculations have been carried out for three slopes (1V:1.5H, 1V:2H and 1V:2.5H) as well as two separate damage criteria (S=2 and 4).

During the initial design, typical cross sections were developed to compare the required armour, underlayer and core volumes for the different slope options. Based on this assessment it was found that the cross sections with the flatter slopes would require larger volumes of core, underlayer and armour rock compared to a comparable cross section with 1V:1.5H slopes. The greater volume of material would equate to a more expensive structure.

Designing for a damage level of S=2 is also recommended as this will minimise the maintenance requirements of the groyne.

As a check, the primary armour size was calculated using Hudson's formula. The results show Hudson's formula indicate slightly larger primary armour would be required on the head of Groyne 4. Hudson's formulae does not explicitly include wave period in the calculation. Due to the relatively short wave periods at the site, Cardno believes that the Van Der Meer formulae gives more accurate estimates of the required primary armour size. It should be noted that the results using Hudson's formulae are very sensitive to rock density, with a density of 2,100 kg/m3 reduces the required primary armour mass by 20% and a density of 2,200 kg/m3 reduces the required primary armour mass by 35%. Both these densities are not uncommon for quarried limestone.

Table 6-6 Groyne 4 Armour mass

		SWAN RESULTS					Armour Mass, M50 (kg)						
	Bed Level	100yr Wave + 100yr WL + 50yr SLR					S=2 (Start of Damage)			S=4 (repairable)			Hudson
Distance from head	m AHD	Hs	H1/10	Hb	Breaking (H1/10>Hb)	Tm01	1V:1.5H	1V:2H	1V:2.5H	1V:1.5H	1V:2H	1V:2.5H	1:1.5H
25m offshore	-1.9	1.98	2.52	2.79	No	5.7	6,097	3,906	2,810	4,001	2,590	1,882	6,200
0	-1.0	1.61	2.04	2.09	No	5.2	3,354	2,158	1,527	2,222	1,427	1,032	3,290
5	-0.8	1.56	1.99	1.92	Yes	5.2	3,203	2,061	1,458	2,122	1,363	986	5,480
10	-0.3	1.39	1.77	1.55	Yes	5.2	2,450	1,609	1,144	1,609	1,063	746	2,880
15	0.2	1.19	1.51	1.17	Yes	5.3	1,769	1,144	810	1,144	746	549	1,240

1 T	3 T	5.5 T	6.6 T

The secondary armour layers should be 1/10th the mass of the primary armour and are presented in Table 6-7.

Table 6-7 Secondary Armour Mass

Primary Armour M_{50} Mass	Secondary Armour M_{50} Mass
6.6T	0.7T
5.5T	0.55T
3T	0.3T
1T	No secondary armour layer

6.3.2 Crest Level

The crest level of the groynes have been set according to the allowable overtopping rates as per the criteria presented in Table 6-3.

Based on the SWAN modelling results wave overtopping discharges were estimated for each combination of wave and water level conditions using the Deltares Neural Network Overtopping Calculator (Van Gent et al 2007).

For each section of groyne, wave overtopping rates were estimated for crest levels of 4, 4.5 and 5m AHD. Waves were assumed to approach perpendicular to the head, and at a 45 degree oblique angle on the trunk.

The results indicate that the criteria that governs the crest level is the criteria for pedestrian safety. To achieve overtopping rates less than 0.3 l/s/m in a 1 year ARI event the minimum crest levels are required:

> **Head of groyne: +5.0m AHD**

> **Trunk of groyne: +4.5m AHD**

The crest level of the groyne can be lowered as the waves get smaller towards shore. The design has included the new trunk section at +4.5m AHD, and then slopes down to +2m AHD at the landward (root) of the groyne where the concrete access path joins the root of the groyne.

For ease of construction the head of the groyne has a crest level of +6.0 m AHD, which is one additional layer of armour rock.

6.3.3 Toe Detail

Given the shallow nature of the site, the toe of the groyne has been designed as an extension of the primary armour layer.

The geophysical investigation indicates that there is generally around 2-3m of sand at the site, limiting potential scour. As the scour potential in deeper water is low, for ease of construction the toe has been designed to sit on the existing seabed where the existing seabed is lower than -1m AHD. The geophysical investigation and aerial images indicate that there is likely to be a limestone shelf underneath the footprint of the groyne which may be at an elevation higher than the base of the groyne along the trunk. The details of this limestone shelf will be confirmed by the contractor prior to starting works. If the limestone shelf is higher than the base of the designed groyne, then the groyne can be constructed directly on the limestone shelf. In this case the bedding layer and geotextile would not be required where the primary and secondary armour is placed directly on limestone bedrock.

Where the seabed is shallower than -1m AHD the toe of the groyne has been designed as an embedded toe to allow for potential scour.

As there is no core or secondary armour layer under the toe, a geotextile filter is proposed to be placed under the bedding layer to stop piping of sand through the toe.

6.3.4 Tie-in with Concrete Access Road

A new concrete access path for Groyne 4 has been designed as part of Stage 3 of the project. Due to grade limitations on the access path and as detailed on the drawings, the concrete access path joins onto Groyne 4 approximately 10m seaward of the root of the groyne. To support the access path and provide scour protection to the road the crest level of the last 10m of the groyne has been raised to match the access path in this area.

This causes the footprint of the groyne to increase over the last 10m of the groyne. Due to the relatively small armour rock requirements in this area. To reduce the overall footprint of the groyne in this area no secondary armour layer is proposed as the armour grading. A smaller toe is also proposed given this section of beach is not predicted to scour below -1m AHD.

6.4 Beach Access Design

As part of the Stage 3 scope of work, concept design of new beach access at Groyne 1 and replacement beach access at Waterland Point was undertaken. As part of the development of the final concept designs, preliminary designs of beach access at both locations were discussed at the Community Reference Group (CRG) meeting held on 8 December 2016. The two beach access locations are discussed separately in the following sections. The City has expressed the preference to undertake the detailed design and construction of beach access structures through a design and construct arrangement.

The beach access structures have been documented to a conceptual level of detail, such that the designs may be developed further as part of a design and construct arrangement with a contractor who is experienced at installing decking structures in a coastal environment. The final staircase designs shall comply with all relevant Australian Standards, and the Contractor shall provide shop drawings for approval prior to proceeding with manufacturing of the staircases.

A handrail and balustrade has been documented for all staircases, in accordance with the requirements of the Building Code of Australia (BCA). A proprietary handrail as supplied by Moddex has been nominated, however alternative handrails may be appropriate should they be proven to be in compliance with the BCA.

6.4.1 <u>Groyne 1</u>

Preliminary concept designs for beach access to both sides of Groyne 1 were developed during Stage 2 of the project and included in Cardno (2016). These preliminary concept designs were used as the basis for a discussion of beach access requirements at Groyne 1 at the CRG meeting held on 8 December 2016. A summary of the outcomes of the discussion at the CRG meeting is:

> There were a diverse range of views expressed by the members of the CRG regarding beach access at Groyne 1, such as:

  o Whether or not a path or boardwalk from the carpark up to Ocean Drive and the Playground in Fred Stubbs Park along the edge of the existing carpark access road was required,

  o Whether or not a beach access structure from the carpark down to the beach was required on both sides of Groyne 1 or just on one side,

  o Whether or not a path/boardwalk structure along the edge of the existing carpark access road should extend around in front of the carpark all the way to a beach access structure to the north of Groyne 1,

  o Whether or not a ramp was preferable/feasible to access the beach south of Groyne 1 from the carpark,

  o Preferences for the choice of decking material for the path and beach access structures around Groyne 1,

> After a long discussion, the following consensus was reached on the following points:

  o A beach access structure to the south of Groyne 1 was more important than access to the north, particularly in light of the results of the modelling of the refined layout in front of the carpark which predict a significantly wider beach in front of the carpark.

  o A path/boardwalk linking the carpark with the footpath adjacent to Ocean Drive and the playground in Fred Stubbs Park was desirable and should be accessible for mothers with prams.

  o The path/boardwalk does not need to continue past the southern side of the groyne and should terminate at this point.

  o The feasibility of the beach access to the south of Groyne 1 being a ramp rather than a staircase should be examined.

- The location of the beach access to the south of Groyne 1 should make careful consideration of the erosion previously experienced in the area and predicted to occur in future severe erosion events.
- A stand-alone beach access structure to the north of groyne 1 should be located within close proximity of the existing toilet block.

> The CRG was unable to reach a consensus position on the following point:

- The choice of decking material for the path and beach access structures around Groyne 1. Preferences for Fibre-Reinforced Plastic (FRP), Composite Decking and Timber material options were expressed by members of the CRG.

Taking into consideration the outcomes of the discussion around beach access at the CRG meeting. Cardno has developed a concept design of beach access around Groyne 1. The following is worth noting in light of the discussion at the CRG meeting.

Upon closer investigation it was found that a ramp structure was not a feasible option for beach access to the south of Groyne 1 for the following reasons:

> The required maximum gradient of a universal access ramp is 1:14 and the elevation difference between the crest of the limestone revetment and the beach in front of it, particularly after an erosion event, is such that a ramp structure would have to be a very long structure to reach sufficient depth with the required gradient. This would require either one continuous ramp section or a switchback-style ramp structure, both of which would occupy quite a large area of the beach.

> Erosion previously observed (and predicted to still occasionally occur with the refined layout) in the area during an extreme erosion event means that the structural design of a large ramp structure would likely be very difficult and significantly more costly than a steeper stair structure.

> The potential for damage to a large ramp structure during an erosion event is substantial which would likely lead to large repair and maintenance requirements.

> The maintenance costs for a large ramp structure located in front of the revetment structures and thus exposed to extreme erosion events would likely be much higher than that of a stair structure due to the greater size of the structure.

> Safety would be an important issue with a ramp which would extend down to a sufficient depth to ensure it was not undermined to the point where there would be a vertical drop between the end of the ramp and the beach level beneath it. There could be potential safety issues with algae and other marine growth on the lower sections if exposed to water for extended periods which can make the ramp surface very slippery and dangerous.

In the concept design, the beach access staircase to the south of Groyne 1 has been located towards the southern end of the limestone revetment which extends between Groyne 1 and the GSC revetment in order to minimise the amount of time that the bottom of the stairs and their usability are affected by erosion. During extreme erosion events the bottom section of the stairs may be reached by the water and/or the beach around the bottom of the stairs may be eroded which may impact the usability of the stairs immediately after the erosion event. The location of the stairs has been selected to minimise this risk and maximise the functionality of this staircase.

The area alongside the existing carpark access road was surveyed after the CRG meeting and it was found that the existing gradient is too steep for a universal access ramp (maximum 1:14 grade) and is steep enough that an elevated boardwalk structure is not feasible. In light of this, the concept design includes a concrete path alongside the carpark access road which will match the existing curb level. The actual beach access stairs will be made of either FRP, Composite Decking or Timber and will tie-in to the concrete path.

As the design requirements do not differ greatly based on the choice of decking material of either FRP, Composite Decking or Timber, the concept design was progressed without a specific material being specified. The choice of material can be finalised during the detailed design process. To aid in the decision-making process the pros and cons of the three material types preferred by members of the CRG are listed in Table 6-8 below.

Table 6-8 Pros and Cons of Beach Access Material Options

Material Type	Pros	Cons
Fibre-Reinforced Plastic (FRP)	Durable, low maintenance, cost effective	Industrial appearance, grating less comfortable on bare feet
Timber	Natural material, aesthetics complement coastal site	Expensive, relatively high maintenance costs
Composite Decking	Durable, low maintenance, cost effective	Solid panels so sand may pile up more, more cost-effective options less aesthetically pleasing

The concept design for the beach access structure to the north of Groyne 1 includes a stand-alone stair structure which leads from the edge of the carpark down to the beach over the informal limestone seawall structure. The location is close to the toilet block as desired by the CRG. Based on discussions with the City after the CRG meeting it was agreed that the width of the beach to the north of Groyne 1 is planned to increase significantly as a result of the proposed changes to Groyne 2 and the associated capital nourishment. If the staircase was constructed before these changes were made, there is a very high likelihood that the staircase would be buried by the increase beach width after the extension of Groyne 2 and associated beach nourishment and would thus be redundant and a waste of money. The need for a staircase north of Groyne 1 after the extension of Groyne 2 and the associated nourishment is uncertain as the increased beach width may mean the public will be able to simply walk from the carpark to the beach down the natural beach profile. Thus it is recommended that the beach access staircase north of Groyne 1 be delayed until after the extension of Groyne 2 and the associated beach nourishment and that the need for a beach access structure in this area be reassessed once the new beach profile has been established and the accessibility of the beach from the carpark can be assessed.

The construction of the beach access around Groyne 1 should preferably be undertaken in early summer to take advantage of the generally reduced beach width in this area during this time of the year. However, the beach access could be constructed at any time of year. The optimum timing and other requirements for construction should be discussed with the design and construct contractor during the detailed design.

To assist the detailed design, a table of design considerations incorporated in to the concept design for the Groyne 1 beach access is presented in Table 6-9.

Table 6-9 Design Considerations for Groyne 1 Beach Access Structures

Item/Parameter	Description/Value
Elevation level of seaward end of staircases	-0.6m AHD (LAT)
Elevation level of kerbing which staircases tie into	+4.38m AHD for northern staircase and +5.84m AHD for southern staircase
Staircase Slope Relationship	Must comply with Building Code of Australia (BCA)
Decking Material	To be decided during detailed design
Geotechnical Information	Cardno is not aware of any geotechnical information relevant to the Groyne 1 beach access locations. If required to undertake detailed design, a more detailed geotechnical investigation should be undertaken along the route of the proposed staircase.
Staircase spanning over limestone revetment	The concept design of the two Groyne 1 beach access staircases incorporates them spanning over the face and toe of the limestone revetment in each location such that the staircase will be supported at the crest and beyond the toe of the limestone revetment
Requirement for design to withstand occasional exposure to waves	The location and elevation of the beach access staircases means they will likely be occasionally exposed to wave action. The detailed design of the staircases and supporting structure will need to take this into account.

The nominated materials and general arrangement for the two staircases near Groyne 1 are similar; a substructure supports a top decking layer. As described above, the decking product for the beach access structures around Groyne 1 is yet to be determined, however the selected product will need to be slip resistant, UV stable and structurally sound.

A concrete footpath is also proposed for the Groyne 1 site. The path follows the back of the existing kerb and therefore matches the gradient of the road at a gradient of approximately 1:10. This is steeper than the requirements of Australian Standard 1428: Access For All, however the site conditions preclude a ramp at a gentler gradient. The beach access staircase to the south of Groyne 1 will be required to meet flush with this new footpath.

6.4.2 Waterland Point

The existing beach access structure at Waterland Point consists of a timber log and chain step structure at the end of a concrete path from the carpark. The existing log and chain structure is in poor condition and has been seriously damaged during extreme erosion events in the past, including the September 2013 storm event. The City has advised that at times the log and chain steps have been near vertical and thus represent a dangerous safety hazard. During a site visit on 19 September 2016, Cardno and City staff observed that the log and chain step structure was in poor condition and it appeared that the public was bypassing the steps and simply walking on the sand to the north of the steps (Figure 6-1).

Figure 6-1 Timber log and chain beach access structure (Photo taken 19 September 2016)

At the CRG meeting on 8 December 2016, a preliminary concept design of essentially replacing the log and chain stair structure with a more solid and robust staircase structure was discussed with the CRG. The members of the CRG agreed with the principle of the preliminary concept design and a consensus preference for it to be made of FRP was agreed. It was noted by the CRG that the stair structure should be made of small mesh size FRP as there have been problems with dog's paws falling through FRP panels with a larger mesh size at other beach access structures and this has been incorporated into the final concept design. An additional advantage of FRP was noted by the CRG in that the mesh allows sand to fall through the structure which will minimise sand build-up on the structure.

The final concept design was developed based on three limiting factors:

> The beach in this area has sustained substantial erosion during historical erosion events and so any permanent access structure would need to extend down to a low enough elevation that the stairs still reach either the sand or the water after an extreme erosion event. The stair structure in the concept design extends down to 0m AHD elevation which is approximately Mean Sea Level.

> The results of the geophysical investigation (refer to section 2.6 of this report) show that there is limestone beneath the beach and dunes near this location and so a permanent stair structure should be founded on this limestone to ensure structural stability during an extreme erosion event but should also not intersect with the limestone layer such that the limestone layer would need to cut into unnecessarily.

> A permanent stair structure should tie-in to the existing concrete access path at some point along the existing path.

Balancing the three limiting factors above, the concept design of the beach access structure at Waterland Point was developed. The FRP stair structure ties in to the existing concrete access path approximately 10m further back up the path than the current end of the path, it touches the approximate position of the limestone layer underneath the beach at approximately +5m AHD, and then extends all the way down to the 0m AHD at the seaward end of the stair structure. The limestone layer is at approximately -1m AHD elevation at the location of the end of the stair structure which is approximately 1m below the stairs, limiting the likely required height of the supporting piles to around 1m. The concept design will require the demolition of a small segment of the existing concrete path in order to facilitate the FRP staircase and will also require some grading of the dunes on either side of the staircase to ensure stabilisation of the dune face. The remodelled dune area will need to be stabilised with matting and revegetated.

Based on the survey of the beach in the area captured in December 2016, most of the stair structure will be buried at the end of construction and should remain so during ambient conditions. However, based on aerial photos of the beach after previous extreme erosion events, it is likely that parts of the stair structure may be exposed by and subject to wave action during such an event and so the structure should be designed to withstand wave action during an extreme erosion event.

The construction of the beach access at Waterland Point should be undertaken in summer to take advantage of the generally more favourable weather and metocean conditions during this time of the year. The optimum timing and other requirements for construction should be discussed with the design and construct contractor during the detailed design.

The nominated materials and general arrangement for the Waterland Point staircase is similar to that at Groyne 1; a substructure supports a top decking layer. The decking material nominated for the Waterland Point staircase is Webforge Mini-Mesh which is a cost effective, low maintenance, slip resistant Fibre Reinforced Plastic (FRP) grating. Other FRP grating products that are equivalent to Webforge Mini-Mesh would also be appropriate for the decking layer.

Table 6-10 Design Considerations for Waterland Point Beach Access Structure

Item/Parameter	Description/Value
Elevation level of seaward end of staircase	0m AHD
Elevation level of concrete path which staircase ties into	Approximately +10.4m AHD
Staircase Slope Relationship	Must comply with Building Code of Australia (BCA)
Decking Material	FRP Mini-mesh (Webforge Mini-Mesh or equivalent)
Geotechnical Information	One profile from the geophysical investigation undertaken as part of this stage of the project is close to the location of the proposed staircase at Waterland Point (Cross-shore Transect 4-4). This information has been used as the basis for the concept design of the staircase. In particular the limestone layer beneath the foot of the staircase is at approximately -1m AHD (approximately 1m below the foot of the stairs), there is a point at approximately +5m AHD where the staircase approximately matches the elevation of the limestone layer and the elevation of the limestone layer east of this point is approximately +6m AHD to where the staircase will join the existing concrete path. If required to undertake detailed design, a more detailed geotechnical investigation should be undertaken along the route of the proposed staircase.
Requirement for design to withstand occasional exposure to waves	The location and elevation of the beach access staircase means it will likely be occasionally be exposed to wave action at its lower elevations. The detailed design of the staircase and supporting structure will need to take this into account.

6.5 Groyne 4 Access Path

As part of the Stage 3 scope of work, the City commissioned Cardno to prepare a detailed design for a permanent access path from the existing carpark to the rear of the proposed Groyne 4. The City requested that this access path be modelled on the concrete access path which was constructed at Groyne 3 after construction of the groyne. The access path for Groyne 4 is intended to be utilised for the construction and future maintenance of Groyne 4, future beach renourishment in the area, as well as pedestrian access to Groyne 4 and the surrounding beach.

6.5.1 Layout

The layout of the Groyne 4 access path was determined based on the location of the existing carpark, the location of the landward end of Groyne 4 and the maximum longitudinal gradient of the path being limited to 15%. It was determined that utilising the existing pedestrian access track was not feasible due to its location and the fact a track which followed this alignment would approach the landward end of Groyne 4 at an oblique angle which would necessitate protection of the lower portion of the access track from scour and wave action (as was done with the Groyne 3 access path). It was thus determined that a different layout was required which approached the landward end of Groyne 4 parallel to the groyne. To comply with the maximum gradient requirement, the chosen layout will require significant excavation into the existing dune and stabilisation and revegetation of the new dune face on both sides of the path will be required.

The section of the new path near the existing carpark will cross over the existing pedestrian access path at two points near the carpark. The design incorporates the new access path replacing the upper section of the existing pedestrian access path which will be demolished and the area rehabilitated. Based on discussions with the City, the intention is for the remainder of the existing pedestrian access to remain functional to the end of its design life and then for it to be reviewed at that point and potentially removed and not replaced.

The layout and design criteria of the concrete access path required a retaining wall to be incorporated into the design along part of the southern side of the path near where the path adjoins Groyne 4. This was required to limit the extent of the required cut areas into Queenscliff Park and avoid disturbing the established trees. The maximum height of the retaining wall has been limited and conservation fencing has been incorporated immediately behind the crest of the retaining wall to minimise safety concerns.

6.5.2 Design Criteria

This section details the design considerations incorporated into the design of the access path to Groyne 4.

6.5.2.1 Gradients

The maximum longitudinal gradient was limited to 15% and the cross-sectional grade is 2%.

6.5.2.2 Earthworks

The layout and levels were determined through consideration of cut/fill requirements, limit to grade of embankments of 1:3, limiting earthworks as much as possible, and minimising disturbance to existing dunes.

6.5.2.3 Durability

6.5.2.3.1 Exposure Classification

Due to proximity to coast and possible direct contact with tidal effects, the concrete access path has been assessed to have an Exposure Classification of C2 in accordance with AS3600. As such, structures and structural elements have been designed for compliance with durability requirements of AS3600 for this exposure classification.

6.5.2.3.2 Concrete Grade

Based upon exposure Classification C2 in accordance with AS3600, Concrete used in the Works is required to be Grade S50 with characteristic strength (f'_c) of 50 MPa.

Based upon AS4997, the following is recommended for structural concrete;

> Special class concrete describing binder type and proportions, water-binder ratio and strength

- Minimum Compressive Strength of (f'_c) 40MPa
- General purpose (GP) Portland Cement alone as a binder or blended cement
- Minimum Cement Content (Portland and blended cement) of 400 kg/m^3
- Drying shrinkage at 56 days not greater than 600×10^{-6}
- Maximum water cement ratio of 0.40 with superplasticisers used to achieve workability.

6.5.2.3.3 Reinforcement

The design has focused on the quantity, distribution and detailing of reinforcement to minimise stresses and cracking and thereby potential for corrosion. A minimum reinforcement bar size of 16mm has been adopted for corrosion in accordance with AS4997.

6.5.2.4 Loading

Based on Table 9.4 of the Rock Manual (CIRIA 2007), the largest primary rock armour to be used in the construction of Groyne 4 (M_{50} of 6.6T) will require a 45T excavator. The concrete path for Groyne 4 was thus designed to handle the live and dead loading associated with this size of excavator in terms of both thickness and width of the track.

6.5.2.5 Drainage

Drainage has been incorporated into the design through consideration of the catchment area, design of the drainage channel for a 5-year ARI rainfall event and the discharge of surface water onto the groyne armour to avoid scour of the beach face.

6.5.2.6 Tie-ins

Tie-in requirements between the access path and the existing carpark, the existing pedestrian footpath and the landward end of Groyne 4 have been taken into consideration in the design.

6.5.2.7 Fencing

Both wind-break fencing and conservation fencing have been incorporated into the design of the Groyne 4 access path.

6.5.3 Design Drawings

The design of the Groyne 4 access path is detailed on the design drawings (see Appendix F).

6.6 Renourishment

6.6.1 Capital Renourishment

Capital renourishment is to be undertaken in conjunction with the construction of the refined layout in order to “fill” the newly created capacity of the compartments between or adjacent to the groynes. The capital renourishment required as part of the refined layout was based on the work undertaken in Cardno (2016) and consists of the following volumes of sand applied to each of the beach sections as per Table 6-11.

Table 6-11 Capital nourishment volumes for refined layout (m^3)

Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	TOTAL
-	3,800	10,600	-	10,800	8,400	33,600

The capital nourishment should be undertaken in conjunction with the groyne construction as per the following:

1. Capital nourishment in Section 6 should be undertaken immediately after construction of Groyne 4,
2. Capital nourishment in Sections 2 and 3 should be undertaken immediately after extension of Groyne 2,
3. Capital nourishment in Section 5 should be undertaken immediately after extension of Groyne 3.

6.6.2 Ongoing Renourishment

Ongoing sand renourishment will be required to maintain the amenity and protective buffer capacity of Quinns Beach after the construction works and associated capital sand renourishment have been completed. This is due to the existing sediment deficit which has been identified and which will not be changed by the construction works and capital nourishment.

Cardno (2015) estimated the existing deficit in the study area to be 20,000 m^3 per annum. This deficit was described as a loss to the nearshore areas during storms, and as a result is not captured in the LITPACK modelling. The offshore movement of sediment is captured in the XBEACH model (refer Chapter 4), however the metocean conditions during that model run are not representative of the normal, ambient conditions and hence the calculated volume of sediment lost offshore in this simulation is expected to be an overestimate of the long term renourishment requirements. A significant percentage of the sediment moved offshore during the XBEACH simulation is expected to move back onshore in the weeks and months following the severe storm event, as is generally observed on most coastlines and has been observed at Quinns Beach in the past.

Following the same approach as that adopted in Cardno (2016), the net volume of sediment lost during the XBEACH simulations above the 0 m AHD contour was calculated over each section for the refined layout. This is presented in Table 6-12. Similarly to the analysis conducted during Stage 2 of the project, the lost sand volume could potentially be reduced (or at least the impact) if the capital renourishment volumes indicated in the LITPACK model runs were included in the XBEACH simulation.

Table 6-12 Estimated sediment loss (m^3) from XBEACH simulation above 0 m AHD contour

Beach Section	Refined Layout
Section 1	4,500
Section 2	5,500
Section 3	7,000
Section 4	2,500
Section 5	2,500
Section 6	1,500
TOTAL	23,500

The total annual renourishment volume for the refined option was estimated using the table above, together with the LITPACK results. This volume is presented in Table 6-13. This volume represents the average total renourishment estimated to be required per annum at Quinns Beach. It should be noted that there are many variables which can affect the annual renourishment volume required in any given year and it is expected that some years may require significantly less renourishment and some years may require significantly more. The volume presented in Table 6-13 is intended for use by the City to set a budget for annual renourishment which should be representative over the long-term. Specific sections should be targeted as required depending on the condition of each beach section at the time of renourishment and the expected changes in the months after the renourishment due to the season of the year in which the renourishment is to be undertaken. It is expected that renourishment on the south side of the groynes will be best undertaken at the start of summer as the beach on the south side of the groynes will generally be at its minimum width at that time of the year. Conversely it is expected that renourishment on the north side of the groynes will be best undertaken at the end of summer as the beach on the north side of the groynes will generally be at its minimum width at that time of the year. In addition to this general expectation, renourishment can be undertaken along any part of Quinns Beach at any time of the year in response to a particular concern. Ongoing monitoring will be essential in this regard to provide information on beach conditions upon which decisions regarding the required volume and placement of sand renourishment can be made.

Table 6-13 Estimated annual renourishment (m^3)

	Refined Layout
Estimated Annual Renourishment	13,000

The cost and efficiency of any beach nourishment activity can be optimised and timed to make use of the natural longshore drift process, and this is something that should be undertaken by the City (with the support of a suitably qualified and experienced Coastal Engineer) each time sand renourishment is required or planned to be undertaken.

6.7 Recommendations on Possible Carpark Seawall

As part of the Stage 3 scope of work, the City requested that Cardno provide advice/recommendations on the potential requirement for a seawall in front of the carpark at Groyne 1 which would provide a protective barrier for the existing carpark against coastal erosion.

As discussed in Cardno (2013), Cardno (2015) and Cardno (2016), an exposed seawall (in the active section of the beach) often suffers from terminal scour and erosion, leading to loss of the beach directly in front and to both sides of the structure. During heavy storm events waves can reflect off the seawall, potentially leading to additional erosion and localised scour directly in front of the seawall. Seawalls can be found at many popular beaches; however they are almost always constructed well back from the existing shoreline, as they are intended to be reached only in extreme storm events, for example the seawall bounding the limit of the beach at City Beach.

An exposed seawall also needs to be keyed in to the surrounding dune system to reduce the risk of scour around the ends of the structure (terminal scour) outflanking the structure. Given the proximity of the car park and foredune at Quinns Beach to the shoreline, it is likely that by the end of summer the seawall will become exposed each year, potentially increasing erosion within Section 3. The seawall will not reduce the local sediment deficit that is present in Section 3 and as such it is likely that over time the seawall will become exposed to a greater degree for an increasing proportion of the year. Loss of the beach and reduction in the visual amenity could be substantial with an uncovered seawall, which would significantly reduce the appeal of this section of Quinns Beach which is the very reason for the carpark in the first place.

The longshore sediment modelling undertaken using LITPACK as part of this stage of the project (refer to section 3 of this report) indicates that once the full construction of the refined layout has been undertaken together with the associated capital renourishment in each section, the beach in front of the carpark should widen significantly from its present condition. With ongoing sand renourishment to offset the sediment deficit in the area, this width should be maintained over the 50-year life of the structure. This wider beach in front of the carpark will act as a protective buffer during coastal erosion events. The XBEACH results predict some erosion on the upper beach face in front of the carpark during an extreme event, however it should be noted that the initial bathymetry of the XBEACH simulation does not include the capital renourishment included as part of the design. Based on the combination of the LITPACK and XBEACH results, Cardno does not expect that a seawall will be required in front of the carpark if the refined layout and associated capital renourishment is constructed and ongoing renourishment is maintained throughout the design life of the structures.

6.8 Recommendations on Possible Groyne 5

As part of the Stage 3 scope of work, the City requested that Cardno provide advice/recommendations on the potential future requirement for a Groyne 5 located somewhere north of Groyne 4, most likely in the vicinity of Waterland Point. A Groyne 5 would only be required if the construction of the refined layout, including construction of Groyne 4, led to future erosion issues to the north of Groyne 4 to such an extent that it was not feasible or became too expensive to manage them using beach renourishment or other “soft” coastal engineering solutions alone.

Based on the results of both the longshore (section 3 of this report) and storm erosion (section 4 of this report) modelling completed to date, Cardno does not expect that a Groyne 5 will be required in at least the short to medium term. However, there are many uncertainties which make prediction of the potential requirement for a Groyne 5 over the long term very difficult to make with any degree of confidence.

Regular monitoring of the section of Quinns Beach north of Groyne 4 will be vitally important after construction of the refined layout and the associated beach renourishment in order to allow the changes in the beach to be monitored and compared to expectations and model predictions. If erosion issues were to be observed to the north of Groyne 4 post-construction of the refined layout, Cardno recommends that the first course of action be beach renourishment and/or other “soft” coastal engineering solutions. Should such action be necessary then continued monitoring of the beach in this area will be essential to monitor the effectiveness of the intervention action and to inform the decision of whether or not it is effectively managing the problem or whether a greater level of intervention, such as construction of a Groyne 5, is required.

The geophysical investigation completed as part of this stage of the project (section 2 and Appendix B of this report) found limestone at quite high elevations (+10 to +15m AHD) at the landward end of all four cross-shore transects at and north of the proposed location of Groyne 4 as well as along almost the full length of the two alongshore transects in this area. As highlighted in section 2, there is one section of alongshore transect 4-2 approximately 30m long which has the top of the limestone layer at lower levels of around +7m AHD. As current mean sea level is approximately 0m AHD, this indicates that the limestone layer is substantially higher than the extreme water levels expected at the site (100-year ARI extreme water level including sea level rise to 50 years is +1.68m AHD). This indicates that the limestone in this section of Quinns Beach is highly likely to provide a substantial degree of protection from potential future erosion to the important infrastructure in this area such as Ocean Drive and so the main driver of the potential need for a Groyne 5 will be beach amenity north of Groyne 4 rather than protection of infrastructure.

6.9 Recommendations on Construction Staging

As part of the Stage 3 scope of work, the City requested that Cardno provide advice/recommendations on staging of the expansion/construction of the three groynes, should the three groynes not be constructed together. From a design perspective, there is nothing preventing the entire construction and renourishment program being undertaken simultaneously (and there are potential cost efficiencies which could be realised by simultaneous construction), however based on discussions with the City it is highly likely that the refined layout and associated renourishment will be spread out over a number of years for non-technical reasons such as budget capacity and other financial considerations as well as minimising the disruption for the public access to one area of Quinns Beach at a time.

It is beneficial to undertake construction in the coastal environment around the summer season when storms and large swell events are less likely and less frequent, seasonal water levels are lower and daylight hours are longer. For these reasons it is envisaged that if the construction program is separated over multiple years, it is likely that the three groynes will be constructed/extended on schedule of one each summer. This approach would mean that there will be two winter seasons during which the refined layout is only partly constructed and it is important to consider the likely performance of a partly constructed layout so that any potential issues can be minimised or avoided.

Based on the above reasoning Cardno recommends that if the groynes are to be constructed/extended separately that they be constructed in the following order:

1. Groyne 4 together with the capital renourishment north of Groyne 4,
2. Groyne 2 together with the capital renourishment on both sides of Groyne 1, then
3. Groyne 3 together with the capital renourishment north of Groyne 3.

The reasoning behind the recommended order is that the significantly larger Groyne 2 is designed to interrupt the longshore sediment transport along Quinns Beach and so if this is constructed first then there are likely to be erosion issues to the north of Groyne 2 in the following winter before Groyne 4 is in place, however Groyne 2 is designed to provide greater protection to the carpark in Section 3 which is a priority for the City and the community. The extension of Groyne 3 is the smallest change of the three and so the absence of this modification for two winters during which the refined layout is partly constructed is not anticipated to have any significant effects.

It should be noted that there is likely to be a benefit to be gained from constructing Groyne 4 as soon after winter as possible to maximise the amount of sand which will be trapped between Groynes 3 and 4 due to the natural seasonal movement of sediment. The same logic applies to Groyne 2.

6.10 Ongoing Monitoring Requirements

As previously outlined in Cardno (2013), regular, ongoing maintenance of the beach along the length of Quinns Beach will be vital to monitor changes in the beach and to inform future decision-making. This will be particularly important after the construction of the refined layout and the associated beach renourishment in order the monitor and track the evolution of the new beach profile as Quinns Beach naturally adjusts to a new equilibrium. Cardno recommends the current monitoring program which the City has been undertaking which includes regularly spaced cross-shore beach profiles along the length of sections 1-6 of Quinns Beach should be continued indefinitely for the time being. Cardno also recommends that the monitoring be conducted more regularly (at least every three months at approximately the middle of each season) during construction and for the first few years post-construction as this will be a critical time in which significant changes are anticipated as the beach adjusts to the new structure layout. As mentioned in Cardno (2013), the key to effective monitoring is consistency, both spatially and temporally, and this principal should be followed by the City with regards to ongoing monitoring of Quinns Beach. Cardno (2013) outlines a comprehensive monitoring approach for Quinns Beach which incorporates photographic monitoring, beach profile surveys, hydrographic surveys, and regular structural inspections, most of which the City can undertake in-house resulting in minimal costs to the City and should be referred to by the City as a guide to future monitoring activities.

7 Uncertainties and Limitations

7.1.1 Climatic Variability

The coastal processes assessment undertaken in Cardno (2015) demonstrated that there is significant inter-annual and decadal variability in metocean conditions which result in significant changes in the overall alignment and sediment storage along Quinns Beach. The time periods investigated with the numerical models in this study as well as Cardno (2015) and Cardno (2016) were selected on the basis of the quality and availability of data to calibrate the models. Offshore measured directional wave data is only available from the Rottnest Island wave buoy from 2004 onwards. Figure 7-1 presents a time series plot of monthly mean wave height, monthly mean alongshore wind stress and vegetation line movements. Whilst the period 2004 through 2014 appears reasonably representative of the 20 years of available measurements, the following differences are of particular note:

> The very stormy winter of 1996, with significant southward wind stress,

> The inclusion of the succession of comparatively reduced northward wind stress (during summer) from 2011/12 through 2014.

Additional discussion is presented in Cardno (2015) and Cardno (2016); however it is important to note that the results presented in this study must be considered within the context of the conditions used to assess the various options for refining the chosen layout option. Whilst appropriate for a comparative study (comparing between options for refining the chosen layout), each option may potentially exacerbate erosion (or accretion) in particular locations when subjected to different climatic conditions.

7.1.2 Climate Change and Sea Level Rise

The Intergovernmental Panel on Climate Change's (IPCC) report AR4 (2007) provided projections for sea level rise based on historical sea level rise and future emission scenarios. Based on the IPCC's projections, the DoT recommended a vertical sea level rise of 0.9m be adopted when considering the impact of coastal processes over the next 100 years (2010 to 2110) (DoT 2010). Hunter's (2009) decadal representation of the recommended sea level rise scenario is presented in Figure 5-1, extended to 2110. These recommendations were formally adopted by the West Australian Planning Commission (WAPC). The more recently updated IPCC AR5 Report (2013) provided updated predictions of sea level rise due to a range of global emissions scenarios and the DoT (2010) recommendation remains consistent with these updated estimates.

The response of Quinns Beach to climate change and sea level rise is likely to be complex and spatially variable due to the complex nearshore reefs and dune morphology. An increase in sea level will potentially lead to an increase in wave energy penetration to the beach and changes to the nearshore wave climate as incorporated into the 50-year wave modelling dataset produced as part of this study. Potential changes to marine habitats due to sea level rise and climate change such as changes in seagrass and kelp concentration and distribution may potentially alter wave penetration through the shallow nearshore reefs to the beach resulting in changes to the nearshore wave climate. For example the recent marine heat wave which occurred in 2011 along the Western Australian coast resulted in significant loss of kelp forests from shallow limestone reefs from Kalbarri through to Lancelin (Wernberg et al 2016). The lost kelp forests have been replaced by small seaweeds which can be generally expected to result in a decrease in the wave dissipation on these shallow limestone reefs. Should a similar change occur at Quinns Beach during the design life of the groynes, for example due to warmer ocean temperatures due to climate change, then the wave energy penetration to Quinns Beach and resulting increase in wave height at the groynes may be greater than that modelled in this study. It is broadly accepted that sea level rise will lead to shoreline recession; however the character of that recession will be a result of the local geology, with the presence and durability of limestone within the dunes at Quinns Beach playing a key role. The Geophysical Investigation undertaken as part of Stage 3 of the project has provided important information on the limestone layers which underlie the beach and dune system along different sections of Quinns Beach and how these hard substrate layers may act as hard barriers to potential future coastal erosion.

In addition to the predicted changes in sea level, climate change may also result in changes in the frequency, intensity, duration and direction of storms that may result in significantly different responses than can be predicted by the use of historical data. These changes in storm parameters are uncertain at this time, and thus cannot be assessed as part of this study.

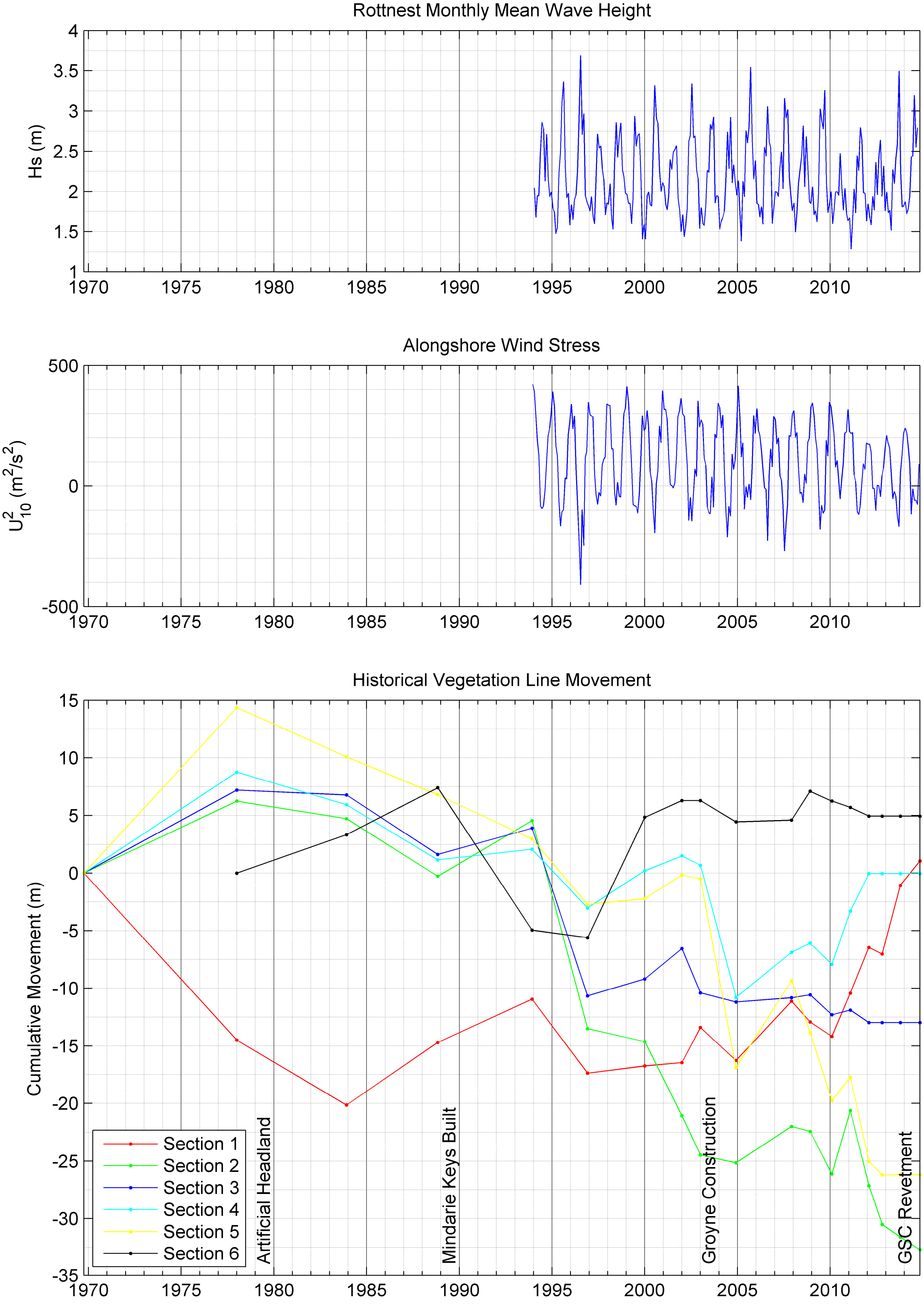

Figure 7-1 Quinns beach measured metocean conditions and vegetation line movement along the study area, plotted cumulatively from 1969 to November 2014

7.1.3 Numerical Model Limitations

Numerical models inherently include assumptions and simplifications in order to model the complex natural world. The LITPACK model is a computationally efficient model that allows simulations of beach morphology to be undertaken over several years (fifty years in this study). LITPACK (and other similar shoreline evolution models) only model longshore transport, in essence assuming that cross shore losses during storms are subsequently recovered post storms over a time scale short enough to avoid significantly influencing supply to the littoral drift process. The model formulations for longshore transport have been developed for long straight sandy coast lines and the influence of coastal structures is incorporated parametrically and assuming no interaction between the structures. Clearly these assumptions do not hold for Quinns Beach, and as such significant model calibration effort was completed in Stage 1 of the study (Cardno 2015) in order to reproduce the observed shoreline changes at Quinns Beach. This study has highlighted the limitations of LITPACK in resolving shoreline changes where multiple coastal structures interact, and as such the results must be carefully considered in the assessment of the various options for refining the chosen layout.

The LITPACK longshore transport model developed for Quinns Beach was calibrated with a focus on the most important areas of interest along Quinns Beach. Some of the model projections presented for the areas of Quinns Beach near the boundaries of the model domain (Sections 1 and 6) differ from recent observed trends (Section 1) or expectations of the influence of new coastal structures based on previous experience at other locations in the Perth Metropolitan area (Section 6). As such the model projections over the long-term for these sections of Quinns Beach should treated with caution.

The LITPACK results presented in section 3 of this report generally project an erosive trend in Section 1 which goes against the recent observed trend of accretion observed since 2010. Conversely, the LITPACK results generally project an accretive trend in Section 6 which is counter to expectations of the potential impact of the construction of Groyne 4 based on the net northerly sediment transport along Quinns Beach. The LITPACK simulation results for these sections of Quinns Beach may be influenced by a number of factors including boundary effects (Sections 1 and 6 are at either end of the model domain and there are no sources or sinks at the boundaries of the LITPACK model), the fact that LITPACK only simulations longshore sediment transport (the XBEACH simulations show that cross-shore sediment transport is important along Quinns Beach), and the model calibration which focussed on the higher value areas of Quinns Beach. The potential implications of these model limitations for these specific areas of Quinns Beach are that the model may be underestimating sediment supply in Section 1 and overestimating sediment supply in Section 6. The result of which could be that the long-term model projections in these areas may not accurately reflect the future morphological response of Quinns Beach to changes to the coastal structures and beach morphology. In these sections the model results should be interpreted as a relative difference between the options rather than a definitive shoreline change.

XBEACH is a 2D morphological model developed specifically to assess the time varying response of coastlines to storm and tropical cyclone conditions. It has specific formulations for dune erosion, over wash and breaching. Whilst the model itself includes fewer assumptions and limitations compared to LITPACK, this comes at the cost of computational expense and significantly greater input data requirements for the set-up of the model. The model requires a significantly more detailed representation of the beach and non-erodible reefs. If gaps in the available input data exist, then significant changes can occur very rapidly as the model adjusts during the model warm-up period. As noted in Cardno (2015), there is limited representation of the nearshore reefs in Sections 5 and 6, which may result in an overestimation of the supply of sand from this area. In addition, the modelling of wave dissipation across the reefs (as discussed in Cardno (2015) is particularly important for Quinns Beach. Additional validation of the wave model in Section 5 was undertaken using additional wave data which was collected by the City offshore of Section 5 during Stage 2 of the project. The results of this validation are provided in Appendix C of this report. As a result of this validation of the wave model, it was decided to keep using the same constant bed roughness map as was applied in the sediment transport modelling in previous stages of the project.

8 Cost Estimate and Bill of Quantities

A Bill of Quantities (BoQ), P90 Cost Estimate and NPV analysis has been prepared by a Quantity Surveyor (Aquenta). This information is detailed in a separate Aquenta report which has been provided to the City.