Local Water Management Strategy

Stage 3 (The Stables) East Port Hedland

Prepared for LandCorp
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Cardno
Shaping the Future

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Executive Summary

This Local Water Management Strategy (LWMS) has been prepared by Cardno (WA) Pty Ltd (Cardno) to support LandCorp's initiative to undertake concept planning and initiate a Scheme Amendment to allow development of Stage 3 (The Stables) East Port Hedland (the Site). The Site is situated in the Town of Port Hedland (ToPH) approximately 5 kilometres east of the town centre and is proposed to consist of a residential subdivision comprised of low and medium density lots (R20 and R40, respectively).

Two indicative development concept options were evaluated for the Site (option 1 and option 2). These options consisted of similar arrangements of residential development, with a primary difference between the two being the location of a caravan park and small differences in R20 and R40 lot area totals.

The development of a LWMS is the appropriate mechanism to establish broad level designs and management measures for flood mitigation and effective stormwater management at the structure planning stage. A LWMS is intended to provide overall guidance to the general stormwater management principles for the area and to guide future Urban Water Management Plans (UWMP) that will support subdivision approval.

This LWMS has been developed to:

> Provide a broad level stormwater management framework to support future urban development;
> Incorporate appropriate Best Management Practices (BMP) into the drainage systems that address the environmental and stormwater management issues identified;
> Minimise development construction costs and ongoing operation and maintenance costs for the land owners and ToPH; and
> Gain support from the Department of Water (DoW) and ToPH for the proposed method to manage stormwater within the development area and potential impacts on downstream areas.

A number of broad level studies that include the Site provide a regional environmental context for the LWMS. These have been reviewed in order to provide suitable background information and provide an indication of the issues requiring further investigation. In summary, the investigations conducted to date indicate that:

> The Site has historically been undeveloped and is used for equine purposes associated with the Port Hedland Pony Club;

> The Site falls south and north from a high ridge line (7.5 to 8.0 mAHD) located approximately 50 m north of Styles Road to 2.5 mAHD in the north and 3.0 mAHD in the south;

> Ground conditions are primarily sandy clay/clay estuarine deposits across the western half of the northern half of site and dune sand on the eastern portion of the site with a limestone ridge spanning the entire southern half of the Site;

> Infiltration has been calculated onsite in two locations to be 5 m/day;

> The estuarine deposits of the western half of the northern portion of the Site are considered to be of High to Moderate Risk of Acid Sulfate Soils (ASS);

> Runoff volumes flowing north towards Pretty Pool Creek that are associated with the six minute 5 year average recurrence interval (ARI) , 10 year ARI and 100 year ARI rainfall events have been modelled as 1,200 m^3, 6,050 m^3, and 11,300 m^3, respectively.

> No surface water was identified in or in the vicinity of the Site;

> Groundwater is present within 3 m of surface;

> Regional groundwater is typically neutral with an average pH of 7.27 and an average TDS of 3,510 mg/L;

> No wetlands or ESA were identified in the vicinity of the Site; and

> No TEC or TPFL are likely to be found within the Site; and

> While no records of species of threatened fauna have been recorded within the Site, three species of threatened fauna have been recorded within 1 km of the Site's boundaries.

The LWMS has determined appropriate water conservation, stormwater management and groundwater management design criteria based on overarching documents, the requirements of the ToPH, DoW and from similar developments.

The overall aim of total water cycle management includes the sustainable consumption of potable water and consideration of all water sources. Therefore the use of water within the development will be minimised wherever possible. This will be achieved through considered landscaping of the Public Open Space (POS) to minimise areas requiring irrigation. In addition, POS areas will be irrigated with fit-for-purpose water where possible. Water efficient appliances and water efficient gardens will be promoted at the lot scale. This will encourage the development to meet the target use of water within household's target of 100 m^3/person/year (Government of Western Australia 2007).

The stormwater management objectives for the Site are to retain (and treat) the first 15 mm of any rainfall event as close to source as possible using soakwells, rainwater tanks or similar where appropriate. It is proposed based on the current masterplanning that runoff from the road reserve will be conveyed to a swale on the northern development boundary via the surface flow on the roads.

The POS areas on the northern boundary will contain the large vegetated swale in order to retain and infiltrate the first 15 mm of a rainfall event from the road reserve. Other strategies to minimise erosion and mitigate sediment transport have also been identified within the LWMS, such as the installation of sediment control devices during construction and the need for an Erosion and Sediment Control Program to be referred to within future UWMP. Specific emphasis has also been placed on the minimisation of erosion and scour associated with fill materials, due to the potential impact of elevated fines deposited in the tidal mangroves of Pretty Pool Creek.

The site model discharges runoff into the swale at multiple locations to more accurately model peak flows associated with rainfall events and ensure that flows are retained and infiltrated. All discharge mechanisms will divert flows away from the identified Aboriginal Heritage sites, and no flow structures are to be built in the vicinity of these sites.

The overall objective for groundwater management is to minimise any changes to the underlying groundwater level and quality as a result of development. It is recommended that prior to commencement of the next stage of the planning process groundwater monitoring is undertaken to characterise annual groundwater fluctuations.

It is proposed that the overall condition of POS areas be monitored on a bi-annual basis following completion of the civil and landscaping works. POS and groundwater salinity monitoring will ensure that the high amenity value of the development is maintained prior to handover of the POS areas to the ToPH.

This LWMS provides a framework that the proponent can utilise to assist in implementing stormwater management methods that:

> have been based on site-specific investigations;
> are consistent with relevant State policies; and
> have been endorsed by the ToPH.

The responsibility for working within the framework established within the LWMS rests with the proponent and their contractors, although it is anticipated the future management actions beyond the proposed management timeframes will be the responsibility of the ToPH.

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1 Introduction

Cardno (WA) Pty Ltd (Cardno) has been commissioned by LandCorp to prepare a Local Water Management Strategy (LWMS) to support concept planning and initiate a Scheme Amendment to allow development of Stage 3 (The Stables) East Port Hedland (the Site). The Site is approximately 35 hectares in area and located in East Port Hedland 9 km north of Port Hedland Airport. The Site is bound by Pretty Pool Creek in the north, a residential development to the east, Styles Road in the south and Cooke Point Drive to the west. The Site location plan is presented in **Figure 1**.

It is important that stormwater runoff from developments is managed and clearly documented early in the planning process. This provides a framework for actions and measures to achieve the desired stormwater management at the subdivision stage. The development of a LWMS is considered to be the appropriate mechanism to establish the concept design and management measures for flood mitigation and effective stormwater management.

The Site is currently zoned 'Rural' and 'Parks and Recreation'. Historical land use has been predominantly undeveloped land along with recreational and equine purposes. Changing land use can have implications for quality and quantity of stormwater generated which can affect the local and downstream environments. In addition, the development of the Site will require the sustainable use of water resources across the wider area and within the Site itself. The overall aim of the LWMS is to ensure that any potential impacts from land use change, and subsequent development, are minimised.

1.1 Policy Framework

There are a number of State Government documents that relate to the Site. These documents include:

> State Water Plan (Government of WA 2007);

> Acid Sulfate Risk Mapping (DER 2006);

> Guidance Statement 33: Environmental Guidance for Planning and Development (EPA 2006);

> State Planning Policy No 3: Urban Growth and Settlement (WAPC 2006); and

> Liveable Neighbourhoods (WAPC 2007).

In addition to the above documents, there are a number of published guidelines and standards available that provide direction regarding the objectives which stormwater management should aim to achieve. These are key inputs and include:

> Decision Process for Stormwater Management in Western Australia (DoW 2009);

> National Water Quality Management Strategy (ANZECC 2000);

> Stormwater Management Manual of Western Australia (DoW 2007); and

> Better Urban Water Management (WAPC 2008).

These guidance documents, together with information from the Town of Port Hedland (ToPH) and Department of Water (DoW), were reviewed to determine the likely data requirements for the Site.

1.2 Sources of Information

A number of broad level information sources that describe the Site have provided a regional context to the LWMS. These were reviewed in order to gather suitable background information for the Site, and also to provide an indication of the issues requiring further and more detailed investigation. The background information was sourced from a variety of references, including:

> DoW's Water Information (WIR) Database Search;

> WA Atlas Database Search;

> Department of Environment and Conservation (DEC) - Contaminated Site Database;

> East Port Hedland Geotechnical Reconnaisance (GHD, 2011);
> Preliminary Environmental Assessment Report (RPS 2011);
> East Port Hedland Concept Plan Report (RPS 2012); and
> Summary of Fatal Flaws for Proposed Development of East Port Hedland Based on Hydrodynamic Modelling.

1.3 Objectives

The LWMS for the Site has been developed to meet the following major objectives:

> Develop a stormwater management strategy for flood protection of the Site and downstream environments;
> Incorporate appropriate Best Management Practices (BMP) into the drainage system to address erosion and sediment transport within the development;
> Develop a water conservation strategy; and
> Gain support from the DoW and ToPH for the proposed method to manage stormwater within the Site and potential impacts on the Site and downstream environments.

2 Pre-development Environment

2.1 Land Use

The Site is approximately 31ha in size and consists of tidal flats and stabilised dunes currently zoned as 'Rural' and 'Parks and Recreation'. The Site has historically been undeveloped, with a small portion used for horse stabling purposes associated with the Port Hedland Pony Club (the Pony Club), which currently occupies approximately 6ha in the middle of the Site.

2.1.1 Indigenous Heritage

Three sites of indigenous heritage significance were identified within the Site during the development of the East Port Hedland Concept Plan. These areas are located along the southern boundary extent and have been identified during consultation with local indigenous communities (Anthropos Australis, 2011). The location of these indigenous heritage sites is outlined in Figure 2.

2.1.2 Non-Indigenous Heritage

Based on examination of the Department of Sustainability, Environment, Water, Population and Communities (DSEWPaC) Heritage Database and the Government of Western Australia State Heritage Inventory no Non-Indigenous Heritage sites have been identified.

2.2 Topography

The Site falls south and north from a high ridge line between 7.5 and 8.0 mAHD located approximately 50 m north of Styles Road. The Site falls to 2.5 mAHD just inside the northern border. From here it slopes into the mangroves and Pretty Pool Creek. A topographic map of the Site and its immediate surrounds can be found in Figure 3.

2.3 Climate

Long term climatic averages indicate that the Site is located in an area of low to moderate rainfall, receiving 332 mm per annum on average (BoM, 2014), with the majority of rainfall received between January and March. The region experiences rainfall on average 33 days a year.

Chart 1 summarises the past 10 years of climate data sourced from the Port Hedland Airport (WA) Bureau of Meteorology station (BoM, 2014), approximately 7.7km from the Site. Temperatures are constant throughout the year and range between approximately 20°C in winter and 45°C in summer. The oscillating trend illustrated in Chart 1 is reasonably constant. This implies that no major changes in temperature have occurred in the last 10 years.

Precipitation trends for the Port Hedland Airport have also remained relatively constant through the previous 10 years. The average precipitation for this period (356 mm) has been relatively constant except for 2013 where there was a significant rise (713 mm). This significant increase in rainfall is attributable to the fact that three of the ten greatest rainfall volumes over the past 30 years occurred in 2013 (BoM, 2014).

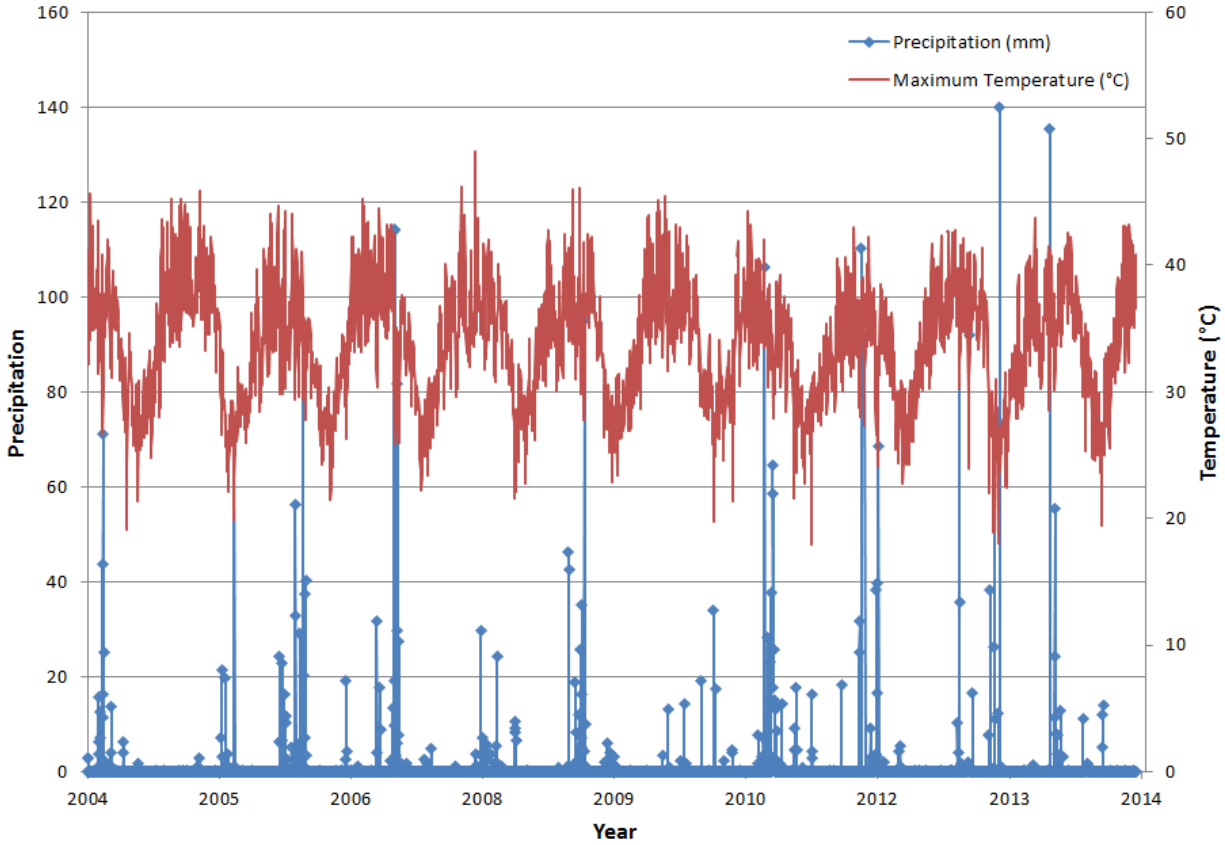

Chart 1 Climate Data for the Port Hedland Airport from 2004 - 2014

2.4 Geotechnical conditions

A geotechnical investigation (Coffey, 2014) of the Site was conducted in order to:

> assess ground conditions;

> characterise the site geotechnically;

> obtain Acid Sulfate Soils (ASS) samples for testing;

> conduct permeability tests; and

> to determine depth to groundwater.

Based on two falling head tests and historic data from the Port Hedland area, an infiltration rate of 5 m/day was recommended for use in the design of soakwells (Coffey, 2014).

2.4.1 Ground Conditions

Ground conditions have been generalised into three areas for subsurface profiles. These subsurface profiles consist of:

> Dune sand – fine to coarse grained sand of loose to medium density located on the eastern half of the northern half of the Site;

> Estuarine deposits – low to high plasticity sandy clay/clay that is typically very soft located on the western half of the northern half of the Site; and

> Limestone – well cemented calcarnite and calcisilite of low to high strength spanning the southern half of the Site.

Figure 4 provides an outline of the soil types based on previous geotechnical investigations (GHD, 2011; Coffey, 2014).

2.4.2 Acid Sulfate Soils

ASS are naturally occurring soils that contain iron sulphide (iron pyrite) minerals. If disturbed by dewatering, drainage or soil excavation, the pyrites can oxidise thereby releasing iron compounds and sulphuric acid. These soils can result in environmental harm and damage to infrastructure. ASS that have been oxidised and resulted in the creation of acidic conditions are termed Actual ASS, and those that have acid generating potential but remain in naturally anaerobic conditions are termed Potential ASS.

ASS are predominantly found in WA's coastal regions in low-lying wetlands and tidal flats and have also been identified inland within WA's South West Region. The potential for ASS to occur within the Site may be assessed by examining the type of soil present and the depth to groundwater. These soils may occur in a variety of waterlogged soils such as dark organic rich soils and muds, peaty wetland soils, some pale grey sands, "coffee rock" (cemented iron and/or organic rich sands) found below the watertable and pyritic soils (Department of Environment and Conservation (DEC), n.d.)

Potential ASS are those which have:

> a pH close to neutral (6.5-7.5);

> contain un-oxidised iron sulphides;

> are usually soft, sticky and saturated with water; and

> are usually gel-like muds but can include wet sands and gravels which have the potential to produce acid if exposed to oxygen.

Actual ASS are characterised by soils that:

> have a pH of less than 4;

> contain oxidised iron sulphides;

> vary in texture; and

> often contain jarosite (a yellow mottle produced as a by-product of the oxidation process).

The DEC provides broad-scale risk maps for several coastal regions of WA where a high or moderate probability of ASS occurrence has been identified. A search of this database has indicated that the low lying estuarine deposits located in the west and northwest regions of the site are classified as having a high to moderate risk of ASS, and the remainder of the site is classified as being of moderate to low risk. Figure 5 demonstrates the ASS mapping for the surrounding area of the site, confirming that the site is within areas of pronounced risk of ASS.

Results of ASS sampling and testing from the site have confirmed that there is a high risk of encountering acid generating soils in the estuarine deposits of the Site, and that the remainder of the Development will be located on subsurface profiles that are possess a moderate to low risk of encountering acid generating soils (Coffey, 2014a).

2.5 Hydrology

A desktop review of available hydrological and topographic information was undertaken to assess the existing hydrological environment. The objectives of this review were to:

> Develop an understanding of how stormwater is currently accommodated onsite;
> Identify impacts of flooding on potential development on current ground elevations;
> Determine if the site acts as a drainage sump to surrounding land; and
> Ensure if the site is acting as a sump to surrounding land that sufficient space is set aside for stormwater in the final plan.

Drainage of the Site currently entails no pipe or pit network, but instead involves overland flow northward to the tidal flats associated with Pretty Pool Creek. The tidal nature of Pretty Pool Creek provides strong tidal influences for drainage and storm surge inundation of the Site.

2.5.1 Storm Surge Inundation

Cyclonic activity impacts the study area during the wet season (between November and May). Due to the proximity of the site to the Pretty Pool Estuary, it is susceptible to ocean inundation resulting from extreme storm surge events. In the future the effects of climate change are likely to cause sea levels to rise. Information provided by Western Australia Planning Commission (WAPC) show sea levels will have risen by 0.9m in 2110, therefore also increasing the likelihood of inundation at the study area.

The WAPC State Coastal Planning Policy 2.6 (WAPC 2003) provides the following recommendation for development in cyclone prone areas: Development should be set back from any areas that would potentially be inundated by the ocean during the passage of a Category 5 cyclone tracking to maximise its associated storm surge.

- (WAPC SPP2.6 Section F.4)

2.5.2 Flooding

As indicated previously, Cardno undertook a flood vulnerability study in 2011 where the 100 year and 500 year average recurrence interval (ARI) flood levels of 5.9 mAHD and 6.6 mAHD were found, respectively. Any development of the site will need the finished floor levels to be above the 100 year ARI flood level by a recommended minimum of 0.5 m. This will require fill over most of the site. No flooding from Pretty Pool Creek was considered in the hydrological model as all modelling was based on pluvial flooding.

Further work undertaken as part of the Mangrove and Erosion Impacts Assessment clarified the proposed finished floor levels for the types of land use proposed within the development as follows:

> Long term accommodation (i.e. residential), assuming a 100 year ARI design life: +6.6 m AHD.

> Short term lease (i.e. caravan park site), assuming a 50 year ARI design life: +5.3 m AHD.

Additional details of the calculations used to determine this level can be found in Appendix A.

2.5.2.1 Mangrove and Erosion Impacts Assessment

Cardno (2015) has undertaken an assessment of impacts to the mangrove community located in Pretty Pool Creek and erosion potential with respect to development of both the Site and the Athol Street development to the north. The result of this assessment has indicated that the development of both sites:

> is likely to have minimal influence on the hydrodynamic regime of the mangrove community under typical tidal conditions;

> could potentially provide favourable colonising conditions for mangrove expansion due to the replacement of high salinity salt marsh with development fill;

> will increase current speeds during extreme events, resulting in the potential for increased erosion potential; and

> the Site will need to be filled to 6.6 mAHD and protected by a rock bund wall for appropriate stability.

The Mangrove and Erosion Impacts Assessment can be found in Appendix A.

2.5.3 Cooke Point Drive Culvert

An existing culvert is located 50 m north of the northwest corner of the Site. The culvert connects an existing drain on the west of Cooke Point Drive to the Pretty Pool Estuary. The drain conveys runoff from a large catchment west of Cooke Point Drive. The Cooke Point Drive culverts consist of two 0.825 mm circular culverts with one way flood gates on the downstream end (Plate 1). Once surface water passes through the culverts, its natural route is northeast towards Pretty Pool Estuary. Flooding from this culvert should not affect the site but will need to be reassessed once planning has been completed (Cardno, 2011a).

Plate 1 Culverts under Cooke Point Drive

2.5.4 Surface Water Quantity

As part of the desktop review, the Site was split into 14 catchments including one external catchment. Catchments 1, 3, 5, 6, 7, 8, 11, 12 and 13 drain north to Pretty Pool Creek. Catchments 2, 4, 9 and 10 drain south from the ridge point towards Styles Road. Catchment 14 is an external catchment located in the residential area to the south east of the Site. The catchment breakup is presented in Figure 6.

The modelling found that currently stormwater flows from the high ridgeline in the south of the Site north towards Pretty Pool Creek. There is also a small amount of runoff which flows south towards Styles Road. In significant storm events there will be sheet runoff across the Site from south to north into the Pretty Pool Creek. Under the current site conditions no runoff is detained onsite. Although runoff from the residential land (Catchment 14) to the south east does flow onto the Site, it flows through Catchment 12 north into Pretty Pool Creek. Flow paths associated with the Site are presented in Figure 6.

The volume of runoff associated with the 6 minute 5 year ARI storm (15 mm) is presented in Table 1. The 5 year ARI storm has served as the design storm for drainage networks in the ToPH, as outlined in Information Sheet 5 – Stormwater Drainage (ToPH, Date Unknown). The 10 year and 100 year ARI event for critical storm duration which the site generates are also presented in Table 1.

Table 1 Runoff Volumes Associated with each ARI event

Flow Direction	6 minute 5 yr ARI (m^3)	10 yr ARI (m^3)	100 yr ARI (m^3)
North to Pretty Pool Creek	1,200	6,050	11,300
South to Styles Road	250	1,050	1,950

The modelling volumes presented in Table 1 give an indication of the amount of water the Site generates in each storm event. With insufficient measures taken during development, these volumes would increase due to the introduction of more impermeable surfaces. It is anticipated that post development runoff generated during these events will decrease due to the requirement of detaining and infiltrating the first 15 mm of any rainfall event. The anticipated volumes provided in Table 1 do not account for the 5 m/day infiltration rate provided by the geotechnical investigation mentioned in Section 2.4.

2.5.5 Surface Water Quality

No surface water was found to be present within the Site.

2.6 Groundwater

2.6.1 Groundwater Levels

As a component of the Site's geotechnical investigation 16 test pits were dug to evaluate near surface ground conditions (Coffey, 2014). Only eight of the 16 pits encountered groundwater, with an average watertable elevation of 1.6 mAHD and an average depth to water of 1.2 m. Surface elevations of the test pits were between 2.4 mAHD and 3.9 mAHD, with one pit located at 6.2 mAHD. Minimal correlation can be inferred between surface elevation and watertable elevation, suggesting that the Site's groundwater is controlled by conditions other than surface elevation.

Greater detail of test pitting conducted can be found in Appendix B.

2.6.2 Groundwater Quality

A query with the DoW's WIR database found 69 boreholes with appropriate water quality indicators within a 20 km radius of the Site. Results received from this query were limited to total dissolved solids (TDS) and pH. The mean, maximum and minimum of these two criteria are tabulated below in Table 2.

Table 2 Groundwater Quality Data within a 20 km radius of the Site

	TDS (mg/L)	pH
Average	3,510	7.27
Maximum	12,932	8.50
Minimum	115	6.74

2.7 Environmental Assets

2.7.1 Flora

A search undertaken of the DER's Threatened Ecological Communities (TEC) and Threatened Priority and Flora (TPFL) database indicated that there are no TECs or TPFLs located within close proximity to the Site.

Currently, no vegetation surveys have been undertaken for the Site. A preliminary environmental assessment report (RPS, 2011) completed for East Port Hedland has determined that it will be necessary to undertake a Level 2 Terrestrial Flora and Vegetation survey in the future to identify if any significant flora and vegetation communities will be impacted by development of the Site.

2.7.2 Fauna

Any native fauna identified to be under threat of extinction, rare, or in need of special protection is provided protection under the Wildlife Conservation Act 1950. Native fauna protected under the Wildlife Conservation Act 1950 is classified as "threatened". The DER maintains a database to help protect and conserve these species and communities which lists taxa that are threatened with extinction as well as taxa that are rare and threatened.

The results of the DER search indicated that there were no records of Threatened and Priority Fauna within the Site, however, Table 3 shows three threatened species of Fauna and 14 species of protected Fauna may be found within 1 km of the Site.

Table 3 Threatened and Protected Fauna

Fauna Name	Latin Name	Conservation Status
Common Sandpiper	*Actitis hypoleucos*	Protected Under International Agreement
Eastern Great Egret	*Ardea modesta*	Protected Under International Agreement
Ruddy Turnstone	*Arenaria interpres*	Protected Under International Agreement
Sharp-tailed Sandpiper	*Calidris acuminata*	Protected Under International Agreement
Sanderling	*Calidris alba*	Protected Under International Agreement
Red-necked Stint	*Calidris ruficollis*	Protected Under International Agreement
Great Knot	*Calidris tenuirostris*	Threatened

Fauna Name	Latin Name	Conservation Status
Greater Sand Plover	*Charadrius leschenaultii*	Protected Under International Agreement
Lesser Sand Plover	*Charadrius mongolus*	Threatened
Airlie Island Skink	*Ctenotus angusticeps*	Threatened
Bar-tailed Godwit	*Limosa lapponica*	Protected Under International Agreement
Rainbow Bee-eater	*Merops ornatus*	Protected Under International Agreement
Whimbrel	*Numenius phaeopus*	Protected Under International Agreement
Grey Plover	*Pluvialis squatarola*	Protected Under International Agreement
Common Tern	*Sterna hirundo*	Protected Under International Agreement
Grey-tailed Tattler	*Tringa brevipes*	Protected Under International Agreement
Common Greenshank	*Tringa nebularia*	Protected Under International Agreement

The search of the DER database is only an indicative assessment of potential communities. A Level 1 Fauna Survey will need to be undertaken to determine if any fauna of significance, or appropriate habitat, will be affected by development of the Site.

In particular, emphasis with respect to the presence of species of turtles should be placed on any fauna surveys of the Site.

2.7.3 Wetland and Sensitive Environment

A review of the DER Wetland Base and the Landgate WA Atlas indicated that there were no geomorphic wetlands of any classification on or in the immediate vicinity of the Site.

According to the DER Native Vegetation Map Viewer no Environmentally Sensitive Areas (ESA) are recorded in the Site.

2.8 Summary of Existing Environment

The pre-development environment of the Site can be summarised as follows:

> The Site has historically been undeveloped and is used for equine purposes associated with the Pony Club;

> The Site falls south and north from a high ridge line (7.5 to 8.0 mAHD) located approximately 50 m north of Styles Road to 2.5 mAHD in the north and 3.0 mAHD in the south;

> Ground conditions are primarily sandy clay/clay estuarine deposits across the western half of the northern half of site and dune sand on the eastern portion of the site with a limestone ridge spanning the entire southern half of the Site;

> Infiltration is expected to occur at a rate of 5 m/day;

> The estuarine deposits of the western half of the northern portion of the Site are considered to be of High to Moderate Risk of ASS;

> Runoff volumes flowing north towards Pretty Pool Creek that are associated with the six minute 5 year ARI, 10 year ARI and 100 year ARI rainfall events have been modelled as 1,200 m^3, 6,050 m^3, and 11,300 m^3, respectively.

> No surface water was identified in or in the vicinity of the Site;

> Groundwater is present within 3 m of surface;

> Regional groundwater is typically neutral with an average pH of 7.27 and an average TDS of 3,510 mg/L;

> No wetlands or ESA were identified in the vicinity of the Site;

> No TEC or TPFL are likely to be found within the Site; and

> While no records of species of threatened fauna have been recorded within the Site, three species of threatened fauna have been recorded within 1 km of the Site's boundaries.

3 Proposed Development

3.1 Development Details

The Site is proposed to be developed into a subdivision containing a combination of low and medium density residential areas (R20 and R40, respectively) in line with the Stage 3 of the East Port Hedland Concept Plan.

Two indicative development concept options (option 1 and option 2) were evaluated in the execution of this LWMS. Anticipated lot provisions for each development scenario are provided in Table 4.

Table 4 Breakdown of Anticipated Lots by Indicative Development Concept Option

Option	R20 (ha)	R40 (ha)	Caravan Park (ha)	Public Open Space (ha)	Road Reserve (ha)	Undeveloped (ha)
Option 1	9.99	4.42	4.14	5.20	7.16	0.54
Option 2	10.30	4.10	4.40	5.20	6.94	0.62

Corresponding site plans for each option's indicative development concept are provided in Appendix C.

3.1.1 Access to Site

Site access for both options will be via Styles Road to the south, with the potential for inclusion of access via Cooke Point Drive.

3.1.2 Location of Public Open Space

Both options outline the same areas for POS, with 5.20 ha of POS broken into six areas as follows:

> Four park areas located on the northern development boundary – 3.05 ha;

> One large park area containing the three identified areas of indigenous heritage – 1.27 ha; and

> The Pony Club – 0.90 ha.

3.1.3 Port Hedland Pony Club

The Pony Club currently occupies approximately 6 ha of land in the center of the Site. Under both options 1 and 2, the Pony Club will be reduced in size to occupy only 0.90 ha on a centrally located plot near the centre of the Site.

3.2 Indicative Development Concept Option 1

The proposed development of option 1 involves the positioning of a large caravan park on the eastern edge of the Site, with a mixture of R20 and R40 interspersed with POS across the remainder of the development.

3.2.1 Caravan Park

The caravan park for option 1 is located in the northeastern corner of the development occupying 4.14 ha of area, and is located on dune sand, as identified in Section 2.4.1.

3.3 Indicative Development Concept Option 2

The proposed development of option 2 involves the delineation of the Site as having the caravan park located on the western edge, with a mixture of R20 and R40 interspersed with POS across the remainder of the site.

3.3.1 Caravan Park

The caravan park for option 2 is located adjacent to the western site boundary. The northern half of the caravan park will be located on estuarine deposits underlying fill, while the southern half will be located on limestone.

4 Design Criteria and Objectives

4.1 Total Water Cycle Management

Total water cycle management recognises the finite limit to a region's water resources, and the inter-relationships between the uses of water and its role in the natural environment. The State Water Plan (DoW 2007) endorses the promotion of total water cycle management and application of Water Sensitive Urban Design (WSUD) principles to provide improvement in the management of stormwater and to increase the efficient use of existing water supplies. Total water cycle management addresses not only physical and environmental aspects of water resource use and planning, but also integrates other social and economic concerns. Stormwater management design objectives should therefore seek to deliver better outcomes in terms of:

> non-potable and potable water consumption;

> stormwater quality management; and

> flood mitigation.

The overall objective for preparing a total water cycle management plan for the proposed development is to mitigate flooding, minimise sediment transport and maintain an appropriate water balance. This objective is central to the LWMS.

4.2 Water Conservation

The overall aim of total water cycle management includes the sustainable consumption of potable water and consideration of all water sources. Therefore the use of water within the development will be minimised wherever possible. The design criteria for water conservation are detailed below:

> Minimise household water usage to meet the target of 100 kL/person/year (Government of Western Australia, 2007);

> Minimise water requirements for the establishment of any vegetated areas;

> Minimise water requirements for the maintenance of POS; and

> Minimise water requirements for swale maintenance.

4.3 Stormwater Management

The overall guiding document for the development of stormwater management strategies is the Stormwater Management Manual of Western Australia (DoW, 2007). The Decision Process for Stormwater Management in Western Australia (DoW, 2009) provides guidance on how urban development can achieve compliance with the objectives, principles and delivery approach outlined in the Stormwater Management Manual of Western Australia.

4.3.1 Stormwater Quality

Water treatment systems and WSUD structures must be designed in accordance with the Stormwater Management Manual of Western Australia (DoW, 2007) and Australian Runoff Quality (Engineers Australia, 2006). Better Urban Water Management (WAPC, 2008) advocates a water quality management principle where existing surface and groundwater quality be maintained as a minimum, and preferably improved prior to discharge from the Site. Through consideration of these guidelines, the primary objective for the Site is to avoid further deterioration of water quality within receiving waterbodies.

The key design criteria which will be adopted to maintain stormwater quality are:

> Treat runoff prior to discharge by detaining the first 15 mm of rainfall onsite as close to source as possible.

> Apply appropriate structural and non-structural measures to minimise the transportation of sediments offsite and reduce applied nutrient loads.

4.3.2 Stormwater Quantity

Stormwater retention and detention structures must be designed in accordance with the Stormwater Management Manual of Western Australia (DoW, 2007) and Australian Rainfall and Runoff (AR&R) (Engineers Australia, 1987). Better Urban Water Management (WAPC, 2008) advocates a water quantity management principle where pre-development peak flows are maintained in the post development environment.

Key design criteria that will be adopted to manage stormwater quantity are detailed below:

> Detaining the first 15mm of rainfall onsite as close to source as practicably possible as per Information Sheet 5 – Stormwater Drainage (ToPH, Date Unknown).

> Ensuring the 100 year ARI event can be contained within the road reserve with a minimum 300mm freeboard to adjacent properties finished floor level.

> No water ponding after 96 hrs to stop mosquito breeding.

4.4 Groundwater Management

The overall objectives for groundwater management are to minimise changes to the underlying groundwater level and quality as a result of development. The design criteria for groundwater management that will be adopted for this LWMS are:

> Minimise changes to underlying groundwater levels as a result of development.

> Ensure that groundwater quality leaving the Site is at least the same, or better, than the water entering the Site.

5 Water Conservation Strategy

The total water consumption associated with the development of the Site can be reduced through the implementation of water conservation measures discussed in the following sections. The conservation strategy has been designed to meet the objectives and criteria stated in Section 4.2.

5.1 Development Water Sources

It is anticipated that development on the Site will access potable water supplies associated with the East Pilbara Water Supply Scheme in line with the recent expansion of the capacity of the Yule and De Grey borefields. Access to scheme water will be via existing Water Corporation assets/infrastructure located along Styles Road.

5.2 Development Scale Water Conservation Measures

5.2.1 Landscaping

There are a number of landscaping design and POS management measures that will be implemented to achieve the design criteria stated in Section 4.2:

> Retention of existing vegetation in newly recreated POS areas to reduce demand for irrigation;

> Retention of native vegetation within POS areas (where possible) to reduce demand for water during establishment;

> Minimal turf will be employed for POS to reduce irrigation demands;

> Turf used in POS will be of a species that requires minimal water and fertiliser; and

> Drainage swales and verge will be vegetated with local planting or minimal lawn where appropriate.

5.2.2 Irrigation

There are a number of irrigation management measures that will be implemented to achieve the design criteria stated in Section 4.2:

> Irrigation systems will be designed and installed according to best water efficient practices;

> Irrigation of revegetated areas within the POS can be established on a two year sacrificial irrigation drip system, to be decommissioned following the establishment of planting; and

> Management of irrigation practices to minimise losses to evaporation.

Conservation of potable water will be encouraged through fit-for-purpose use in order to minimise any water waste. Through fit-for-purpose use the irrigation of POS and landscaped areas can be undertaken using groundwater, treated wastewater and/or greywater.

A search of the DoW's Water Register has indicated that there is sufficient groundwater allocation available for the site to use in irrigation, however, sufficient sampling and testing of the groundwater source to be used will be necessary to ensure its suitability for use due to the coastal nature of the Site.

5.2.3 Community Awareness and Education

Landowners shall be provided with reputable reference material at the point of sale from sources such as the Water Corporation's Waterwise Program (2011), and the Your Home initiative (Commonwealth of Australia 2011). This information will cover a number of topics including:

> Grey Water Recycling;

> Sustainable landscaping and water efficient gardening;

> Water conservation in the home; and

> Sediment control and erosion mitigation on Lots.

5.3 Lot Scale Water Conservation Measures

5.3.1 Potable Water Supply

Scheme water for the area will be sourced from the water pipe network via the existing mains on Styles Road. Water Corporation manages both the distribution and reticulation pipe network infrastructure within the ToPH.

It is proposed that POS areas source any required irrigation from fit-for-purpose sources, and, as such, potable water will only be supplied to the development for usage within the lots.

5.3.2 Alternative Water Supply

Potable scheme water supplied by Water Corporation can be conserved by utilising low quality water, such as greywater, for uses that do not require water of a higher quality. While greywater recycling systems will not be mandated within this LWMS, landowners will be made aware of the benefits of these systems at the point of sale.

5.3.3 Water Efficient Appliances

Significant reductions in water uses can be achieved with the use of water efficient appliances. Table 5 gives an example of the water uses of typical appliances versus water efficient appliances (Australian Government 2009 and Melbourne Water 2003). These water use rates have been used in the water balance investigation.

Table 5 Water Efficient Appliances

Appliance	Water Consumption (kL/year)	
	Standard Device	Water Efficient Device
Toilet	12 L/flush	4 L/flush
Washing Machine	130 L/wash	40 L/wash
Shower Head	15 - 25L/minute	6 - 7L/minute
Taps	15 - 18L/minute	5 - 6L/minute

The water conservation strategy proposes all lots use water efficient appliances. Water efficient shower heads and tap fittings are already mandated as part of the Building Code of Australia (ABCB 2011), however, although not mandated, the uptake of other devices will be encouraged through education from the developers at point of sale.

5.3.4 Water Balance

A potable water balance based on general assumptions (Appendix D) was conducted to determine the effectiveness of the water conservation strategy for both conceptual development scenarios outlined in Section 3.1.

Due to the conceptual nature of the proposed development at this stage, the water balance has been based on the rates and calculation methodology presented in the Water Corporation Spreadsheet for H_2Options, provided in Appendix D. The exception to this methodology was that the household types were changed from “Traditional” and “Terrace” to R20 and R40, with

The resulting water consumptions for each development option are provided below in

Table 6.

Table 6 Water Consumption Requirements by Conceptual Development Option

Option	Drinking Water (ML/year)	Non-drinking Water (Ml/year)	Development Total (ML/year)	Per Person (m^3/year)
Conceptual Development Option 1	45.6	48.7	94.2	100
Conceptual Development Option 2	45.9	55.7	101.6	109

5.4 Wastewater Management

The wastewater management strategy for the Site is to pipe wastewater from lots to the existing ToPH sewerage system which is treated at the South Hedland Wastewater Treatment Plant.

Further investigation and design of the wastewater system for the Site should be explored during the UWMP phase.

6 Stormwater Management Strategy

6.1 Proposed Stormwater Management Plan

Surface water runoff will be managed both on a development scale and a lot scale. The principles behind the stormwater management strategy are:

> to detain the first 15 mm of rainfall on lots at the lots in soakwells, rainwater tanks or similar, as best suits the ground conditions of the eventual design;

> to detain the first 15 mm of rainfall from the road reserve in a swale located on the northern boundary of the Site; and

> to convey all additional rainfall northward to Pretty Pool Creek via the road reserves and swale.

Other strategies to minimise sediment transport are discussed in the following sections. The drainage system has been designed to achieve the objectives and criteria stated in Section 4.3 and has been applied against the two indicative development concept options described in Sections 3.2 and 3.3.

6.1.1 Aboriginal Heritage Sites

Three sites of recognised aboriginal heritage are located within the indicative development boundary. The proposed stormwater management strategy will facilitate the drainage of excess stormwater away from these areas in order to protect them from any damage from inundation and erosion. At present, the stormwater management plan does not utilise any drainage structures on, or in the immediate vicinity of, the aboriginal heritage sites.

It should be noted that detailed design of the development stormwater drainage system will be undertaken at the UWMP stage. At this time, the proposed strategy may change, however, the need to protect these sites should be maintained.

6.2 Stormwater Management Strategy

The stormwater strategy for the Site is to detain the first 15mm of rainfall onsite within lots through soakwells, rainwater tanks or similar.

All other flows will be conveyed across the catchments using the roadways to a swale situated along the northern site boundary. The swale will discharge any rainfall events greater than 15mm north into Pretty Pool Creek. Discharge from the swale will occur via overtopping along the entire length of the swale as a means to reducing scour. Design of this swale was undertaken on a catchment scale in order to ensure that individual regions of the swale adequately detained catchment specific peaks. It is noted that the swale may need to have a low flow discharge outlet depending on ground conditions, which will be based on the final masterplan design.

6.3 Post Development Stormwater Modelling

Modelling of the post development environment for both development options outlined in Section 3.1 has been undertaken using XPSWMM in order to demonstrate the performance of the proposed drainage strategy. These models were built to characterise the hydrological behaviour of the post development environment for each option. Each option involved the division of the Site into 11 catchments as demonstrated in Figure 7 and Figure 8. Modelling parameters and assumptions are provided in Appendix E.

6.3.1 Post Development Catchments

The post development was modelled as 11 subcatchments for both indicative development concepts (see Appendix C).

For both scenarios modelled, an additional catchment external to the development boundary has been included. This catchment (Catchment K) is located to the southeast of the indicative development boundary and contains the area of the existing developed lots east of the Site that will drain northward through the Site's eastern extent via the swale.

Catchment D in both scenarios contains approximately 5.20 ha of POS which contains three sites of aboriginal heritage. Catchment H in both scenarios contains approximately 0.89 ha of POS which it is understood may be allocated to the Pony Club.

For option 1, Catchment I contains 4.2 ha of area delineated as the preferred location for the proposed caravan park. For option 2, Catchment A includes a slightly larger caravan park area at 4.3 ha.

The remainder of the catchments, and the remaining areas of both catchments H and D, are developed lots with mixtures of both low density (R20) and medium density (R40) residential lots. Regardless of development scenario, drainage from each catchment will be northward to Pretty Pool Creek.

6.3.2 15mm Rainfall Event

6.3.2.1 Lots

Runoff generated from the first 15mm of a rainfall event on lots has been modelled to be detained onsite through soakwells, rainwater tanks or similar, whatever is most applicable to the final design. For the purposes of the LWMS volume calculations have been undertaken for lot soakwells. It is acknowledged that it may not be possible to use soakwells at all locations across the site . As such, should infiltration not be possible, rainwater tanks, or a similar device, could be implemented. The volumes required within this type of system would be subject to change.

The modelled volume of each soakwell has been standardised for both conceptual development options. Soakwells for development option 1 have been standardised to approximately 1.13 m^3 per 100 m^2 of impermeable lot area and soakwells for development option 2 have been standardised to approximately 1.27 m^3 per 100 m^2 of impermeable lot area.

Soakwells have been designed based on the assumption of drainage from all sides of the soakwell and an infiltration rate of 5.0 m/day[1], based on the recommendations of the geotechnical investigation (Coffey, 2014).

Design parameters for the soakwells of options 1 and 2 are outlined in Table 7 and Table 8, respectively. These comply with the strategy outlined in Section 4.3.

Table 7 15mm Required Soakwell Detention for Option 1

Catchments	Soakwell Volume (m^3)	Volume of Rainfall (m^3)	Number of Lots	Infiltration rate (m/day)
A	2.5	118	40	5.0
B	2.5	164	68	5.0
D	2.5	177	66	5.0
E	2.5	226	69	5.0
G	2.5	48	26	5.0
H	2.5	345	122	5.0

[1] This is based on permeability testing undertaken by Coffey 2014 on site at two locations. It is noted that fill imported to the site or existing ground conditions across the site may have a different permeability and as such it is recommended that soakwell calculations are revisited when the *in-situ* permeability of each location has been determined.

Table 8 15mm Required Soakwell Detention for Option 2

Catchments	Soakwell Volume (m^3)	Volume of Rainfall (m^3)	Number of Lots	Infiltration rate (m/day)
B	3.0	68	18	5.0
D	3.0	220	90	5.0
E	3.0	227	69	5.0
G	3.0	48	26	5.0
H	3.0	313	113	5.0
I	3.0	188	63	5.0

For lots with insufficient drainage to facilitate the installation of soakwells, rainwater tanks will be employed to retain the first 15 mm of rainfall on the lot. For both options 1 and 2, rainwater tank sizing will be dependent on lot size. R20 lots will require rainwater tanks with an average volume of approximately 3.75 m^3, and R40 lots will require rainwater tanks with an average volume of approximately 1.80 m^3. These sizes comply with the strategy outlined in Section 4.3

6.3.2.2 Roads and Caravan Park

Flows from the roads and the proposed caravan park will be conveyed to the swale for treatment. The proposed swale will have a side slope of less than 1:3 (as per Chapter 9 DoW, 2007) and on average will be 0.3 m deep. Preliminary modelling has shown the swale to be sufficient to contain the first 15 mm of rainfall, with a maximum drainage time for both options 1 and 2 of approximately 3 hours, based on an infiltration rate of 5.0 m/day. This assumes a permeability of approximately 5 m/day. At the detailed design stage it is strongly recommended that *in-situ* permeability testing is undertaken to confirm the permeability rate prior to design of the final retention feature.

6.3.3 Rainfall Events Greater than 15 mm

Stormwater runoff from lots will be directed towards the road network which drains towards the swale system. When capacity of the swale is reached (i.e. after the first 15 mm of rainfall), it is anticipated that the presence of the pathway will serve as a spillway crest for the swale allowing for drainage to occur across the majority of the swale's crest towards Pretty Pool Creek.

At the UWMP stage more extensive modelling should be undertaken to evaluate the operation of the swale system proposed.

6.4 Stormwater Quality Management

Management of erosion and sediment transport within the Site must occur at all levels of planning from pre-construction until handover to the ToPH. Strategies that could be adopted to minimise erosion and control sediment transport prior to and during construction include:

> Retention of vegetation along the road verge;

> Incremental clearing of the development in stages to minimise erosion opportunities;

> Ground disturbance activities avoided during the wet season;

> Temporary offline sedimentation basins utilised, if required, to collect fine sediments in the event that drainage from the stage being developed cannot follow the drainage strategy described in Section 4.3.2;

> Revegetation to occur as soon as possible; and

> An Erosion and Sediment Control Program documented for the development.

Long term stormwater quality management within the Site will occur within the swale areas. This can be achieved through the swale design including erosion and sediment control features such as vegetation and rock armour.

7 Groundwater Management Strategy

7.1 Groundwater Level Management

The objectives for groundwater management are to maintain the groundwater level and quality in the post development environment.

Indicative groundwater levels for the site have indicated that groundwater is located, on average, at approximately 1.6 mAHD (Coffey, 2014). An average of 3.0 m of fill is required onsite to raise the development out of the floodplain to 6.4 m as required by the Coastal Vulnerability Study (Cardno, 2011). This fill, combined with a maximum indicative groundwater elevation of 2.7 mAHD, indicates that there is likely to be a minimum of approximately 3.7 m between the new ground surface and the groundwater level. It is recommended that a groundwater investigation and subsequent monitoring plan be undertaken for the site to determine typical groundwater ranges and how the development may be impacted prior to detailed design.

7.2 Groundwater Quality Management.

As stated in Section 4.4 the overall aim for groundwater is to ensure that the quality leaving the Site is at least the same, or better, than the water entering the Site. This will be achieved by the use of swales throughout the Site to treat the first 15mm of rainfall runoff from the roads and on lot detention. The use of swales will ensure groundwater is protected from pollutant transport such as hydrocarbons. Any pollutants or contaminants the new development may produce are not likely to infiltrate to the groundwater. It is recommended that concurrently with the groundwater level monitoring that site specific groundwater quality monitoring is also undertaken.

8 Management and Maintenance

The design and construction of the stormwater system has been undertaken in a manner that promotes the long-term health of the WSUD. Additionally, these areas often require active ongoing management, particularly in the first years after construction to ensure that the features continue to provide the designed functions.

An effective Management and Maintenance Plan (MMP) will also incorporate an effective monitoring regime to provide guidance to the required level of intensity of management actions. A MMP can provide guidance of the actions required to ensure that the overall objective is met. The overall objective of the MMP is to:

> *Maintain amenity and stormwater functions of the vegetated basins and swales whilst minimising potential environmental impacts and disturbance to surrounding residents in the longer term, and to ensure that the system is in an appropriate and sustainable condition at the point of management handover.*

The overall objective will be achieved through the implementation of a number of management actions that will be carried out at regular intervals for a period of two years from practical completion of the swale. The key areas that will be addressed through the implementation of this management plan are:

> nutrients and water quality;

> gross pollutants and sediments; and

> vegetation.

8.1 Nutrients and Water Quality

8.1.1 Structural Measures

Structural measures proposed within this LWMS maximise the removal of nutrients from stormwater flows. The designed stormwater system provides detention and treatment of the first flush rainfall through the use of vegetated swales. The combination of these components provides primary and secondary treatment to the stormwater discharging from the Site.

The actions to be implemented are detailed in Table 9.

8.2 Gross Pollutants and Sediments

8.2.1 Structural Measures

Gross Pollutants (GP) can potentially introduce health risks and reduce the overall visual amenity of an area. Sediments can carry nutrients to downstream waterbodies and clog up stormwater structural measures, in particular during the construction stage, preventing the system from working efficiently. Straw bale barriers can be used on the downslope side of lots and road reserves to prevent sediment being transported onto the road reserves and flushed towards the stormwater drainage system and the mangroves of Pretty Pool Creek. Vegetation in the swales can be used to trap GPs and sediment which can be removed manually as part of the management plan.

8.2.2 Non-Structural Measures

While the swales and basins will trap the collected GPs, ongoing management and maintenance of GPs will include:

> Periodic visual inspection of the swales.

> Removal of GPs to an offsite disposal facility in response to observations.

> Provide street sweeping to remove sediment-bound nutrients prior to runoff into swales.

The actions to be implemented are detailed in Table 9.

8.3 Vegetation

8.3.1 Weeds

Heavy growth of aquatic and terrestrial weeds can impair the aesthetic value and hydrological functioning of the swales. The primary means of monitoring and detecting weed growth will be regular visual inspections by maintenance contractors. Management of weeds will therefore include:

> Visual monitoring of the swale for presence of weeds. The information gained will then be used to direct the need for any remedial actions such as:

- Manual removal of weeds as deemed necessary.
- Application of approved herbicides (Round-up, Fusilade or similar) to terrestrial weeds.

The actions to be implemented are detailed in Table 9.

8.3.2 Infill Planting

Experience with managing other developments has shown that some plants are subject to theft and vandalism. Additionally, there is the potential for plants to perish prior to establishing deeper root systems. To manage this potential issue, infill planting will be conducted to maintain the required plant densities as per future landscape plans. Management of infill planting will include:

> Visual inspections of the swales for infill planting requirements. The information gained during inspections will be used to guide the need for infill planting.

> Conduct infill planting.

The actions to be implemented are detailed in Table 9.

Table 9 Maintenance Schedule and Responsibility for Management Actions

Action	Timing	Location	Responsibility
Harvest of nutrient removing vegetation	As required	Swales	Maintenance Contractor
Install straw bale barriers	During construction of the stormwater drainage network	Downward slope of all lots and road reserve	Civil contractor
Construct swales	During construction of the stormwater drainage network	Swales	Civil contractor Landscape contractor
Inspect for GPs and sediments	Minimum three-monthly	Swales	Maintenance contractor
Remove GPs and sediments	In response to observations	Swales Offsite disposal facility	Maintenance Contractor/ ToPH
Dispose of waste to an approved facility	Following removal of GPs		
Provide street sweeping	Monthly – Especially during the building phase	For entire development site	
Visually monitor for terrestrial weeds	Three-monthly basis	Swales	
Manually remove weeds	In response to visual inspections	Swales	Maintenance Contractor/ Proponent
Apply herbicide to weeds at manufacturer's recommended rates	In response to visual inspections Three-monthly basis		
Visually monitor for infill planting requirements			
Conduct infill planting	In response to visual inspections		
Provide information to residents	At point of sale		

9 Monitoring

9.1 Groundwater Monitoring

Pre-development monitoring of groundwater levels and quality across the Site will need to be undertaken over two wet seasons in order to establish baseline conditions and trigger levels.

As a requirement of Better Urban Water Management (WAPC, 2008), and due to the proximity of the Site to the potentially sensitive habitat of the mangroves located in Pretty Pool Creek, post development monitoring of groundwater level and quality should be undertaken. The specifics of the post development groundwater monitoring regime should be proposed during the UWMP stage once pre-development monitoring has been completed and groundwater levels are better understood.

9.2 Maintenance of Fill and Swale System

The tidal nature of Pretty Pool Creek will require monitoring of fill materials used on site to ensure no significant erosion is being undertaken due to tidal influence and that erosion prevention measures are functioning as intended.

Post development maintenance of the drainage swales surrounding the Site will be required to ensure the erosion and scouring measures are functioning as intended. The proponent will be responsible for maintenance and erosion control of the surrounding drainage network for a period of two years following completion of the development.

A visual assessment will be undertaken on a bi-annual basis to monitor the condition of swales to ascertain that the maintenance activities specified within Section 8 achieve the objectives of the MMP. If the results from the annual monitoring report indicate that action is required to address an issue, a number of contingency measures can be employed (see Table 10).

Table 10 Visual Assessment and Contingency Actions Plan

Aspect to Monitor	Trigger for Action	Contingency Action
Debris in drainage system	Visual assessment finds the condition of an aspect poor as compared to the initial visual assessment undertaken at completion of construction.	Remove debris
GP litter		Remove litter
Storm damage		Repair drainage system to original condition
Silt		Remove silt build up and restore to original condition
Weed infestation		Removal of weeds
Condition of paving		Restore paving to original condition.
Indicators of theft/vandalism		Restore to original condition by taking appropriate action on a case-by-case basis.
Litter		Remove litter.

10 Requirement for an Urban Water Management Plan

The requirement to undertake preparation of more detailed water management plans is generally imposed as a condition of subdivision. The development of the UWMP should follow the guidance provided in the Urban Water Management Plans: Guidelines for Preparing Plans and for Complying with Subdivision Conditions (DoW 2008).

While strategies have been provided within this LWMS that address planning for water management within the Site, it is a logical progression that future subdivision designs and the supportive UWMP will clarify details not provided within the LWMS. The main areas that will require further are detailed in the following sections and include:

> Modelling of the local drainage network.

> *In-situ* permeability testing.

> Configuration of treatment and detention swales.

> Implementation of water conservation strategies.

> Non-structural water quality improvement measures.

> Management and maintenance requirements.

> Construction period management strategies.

> Monitoring and evaluation program.

In addition, all infiltration systems should be shown to empty within the designated times as detailed in Water Sensitive Urban Design: Basic Procedures for 'Source Control' of Stormwater (University of South Australia, 2008).

10.1 Modelling of Local Drainage Network

It is acknowledged that the drainage strategies documented in this LWMS are based on broad assumptions and data. These assumptions are considered adequate for development of the proposed swale sizes and of an appropriate level of detail. However, verification of proposed subdivision drainage designs should be undertaken by modelling the detailed drainage design. These detailed drainage designs should include road designs that show longitudinal grades to ensure sufficient capacity to contain stormwater without causing a flood risk to the development. In addition, the drain time of all WSUD features proposed should meet the 96 hr design requirement. Such modelling will allow verification that development undertaken is consistent with the design criteria given in Section 4**.**

10.2 *In-situ* Permeability Testing

Prior to any detailed design, Cardno strongly recommends that *in-situ* permeability testing is undertaken across the site and particular within the areas proposed for retention storage. This will provide site specific, location specific results for final design modelling to ensure sufficient capacity for storage has been provided.

10.3 Configuration of Treatment and Detention Swales

While the drainage catchments have been defined based on the current plans and available information, it is possible that these could undergo some change to accommodate stakeholder feedback prior to final subdivision design.

The exact location and shape of drainage features will be specified and presented within the future UWMPs. In order to review the final configurations, the hydraulic model that has been developed to support this LWMS may need to be refined. It is expected that the vegetated swales will be designed to a level that provides detailed cross-sections, sizes of detention and storage areas, detained volumes, culvert sizes,

longitudinal grade, inverts etc. Onsite permeability testing will be carried out to aid stormwater modelling. The ultimate aim of revising the model will be to confirm that the final detailed drainage design meets the design criteria and drainage strategy presented in this LWMS.

10.4 Water Conservation Strategies

A number of potential measures to conserve water have been presented in this LWMS. Landscape design measures that will be incorporated into the water conservation strategy should be further detailed within future UWMPs. The manner in which the developer intends to promote water conservation measures discussed in this LWMS to future lot owners should also be discussed within future UWMPs.

10.5 Non-structural Measures

Guidance for the development and implementation of non-structural water quality improvement measures is provided within the Stormwater Management Manual for Western Australia (DoW 2007). Some measures will be more appropriately implemented at a local government level, such as street sweeping, however many can be implemented relatively easily within the design and maintenance of subdivisions and the swales. These measures are expected to be detailed within future UWMPs.

10.6 Management and Maintenance

The management measures to be implemented address surface water quality, such as the use of vegetation in swales, will require ongoing maintenance. It is therefore expected that the future UWMPs will provide detailed MMPs that will set out maintenance actions (e.g. weeding), timing (i.e. how often it will occur), locations (i.e. exactly where it will occur) and responsibilities (i.e. who will be responsible for carrying out the actions) based on the proposed landscape plans. Given that approval from the ToPH and DoW will be sought for the proposed measures, it is anticipated that consultation with these agencies will be undertaken and referral to guiding policies and documents will be made.

10.7 Construction Period Management Strategy

It is anticipated that the construction stage will require some management of various aspects (e.g. sediment, dust, surface runoff, noise, traffic etc). In particular, sediment transport and dust generation must be minimised during construction works.

Measures to control dust generation during construction may include:

> Not undertaking earthworks during dry, windy conditions.

> Water down cleared areas will occur as necessary during dry dusty periods.

> Covering materials during construction to reduce dust emissions.

Measures to prevent erosion and minimise sediment transport during construction must be documented within an Erosion and Sediment Control Program and can include a number of measures as stated in Section 8**.**

10.8 Monitoring

It will be necessary to confirm the management measures that are implemented are able to fulfil the intended management purpose, and are in a satisfactory condition at handover to the ToPH. A monitoring program should be developed to provide this information, and it should include details of objectives of the monitoring program, relevant issues and information, proposed methodology, monitoring frequency and reporting obligations. The monitoring identified in Section 9 will be further detailed at the UWMP stage.

A summary of the objectives a UWMP would need to comply with is detailed below in Table 11.

Table 11 UWMP Objectives

Objective	Requirement
Modelling of Local Drainage Network	Confirmation that 15 mm event is contained within the swale system
	Confirmation that 100 yr ARI event is contained within the road reserve
	No water ponding after 96 hrs
Configuration of Detention Areas	Detailed design of drainage features including: cross-sections, sizes of detention swales, detained volumes, culvert sizes, longitudinal grades, inverts etc.
Water Conservation Strategies	Landscape design measures to enable water conservation and method of promoting water conservation to future lot owners
	Detailed design of landscaping for vegetation water requirements and to enable better scoping of water sourcing
Non-structural Measures	Update of volumes required for irrigation from any potential changes to the Draft Structure Plan
Management and Maintenance	Management and Maintenance Plans to include: maintenance actions, timing, locations and responsibilities
Construction Period Management Strategy	Erosion and Sediment Control Program to include measures to prevent erosion and minimise sediment transport during construction.
Monitoring	Monitoring Program to provide details of the relevant issues and information, proposed methodology, monitoring frequency and reporting obligations

11 Implementation

11.1 Roles and Responsibility

This LWMS provides a framework that the ToPH can utilise to assist in implementing stormwater management methods that have been based on site specific investigations, are consistent with relevant State policies and have been endorsed by the ToPH. The responsibility for working within the framework established within the LWMS rests with the proponent and contractors, although it is anticipated future management actions beyond the proposed management timeframes will be the responsibility of the ToPH.

11.2 Assessment and Review

Reporting to the ToPH will occur annually, detailing the monitoring performed to date. This encompasses the visual/qualitative assessment of the overall condition of the development. At the end of the two year monitoring and reporting period, the overall condition of the swale will be assessed and the condition reported to the ToPH within the final monitoring report.

The overall criteria for successful completion and establishment of the area will be to fulfil the intended purpose of providing a stormwater attenuation function and increasing the overall visual amenity of the Site in general. If the swale fulfils the stated objectives, the Site will be considered to be complete and in a suitable condition for management handover to the ToPH.

If, at the end of the two year monitoring and reporting period, the drainage features are not considered to fulfil the management objectives, the proponent will work with the ToPH to select appropriate contingency actions that will aim to achieve a mutually satisfactory outcome.

12 References

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13 Abbreviations

A	
AHD	Australian Height Datum
ARI	Average Recurrence Interval
ASS	Acid Sulfate Soil
ATU	Anaerobic Treatment Unit
B	
BMP	Best Management Practices
D	
DEC	Department of Environment and Conservation
DoW	Department of Water
F	
FSA	Flood Storage Basin
G	
GP	Gross Pollutants
H	
Ha	Hectares
L	
LWMS	Local Water Management Strategy
LSP	Local Structure Plan
M	
mAHD	metre Australian Height Datum
MGL	Maximum Groundwater Level
MMP	Management and Maintenance Plan
P	
PASS	Potential Acid Sulfate Soils
POS	Public Open Space
T	
TEC	Threatened Ecological Communities
TPFL	Threatened Priority and Flora
ToPH	Town of Port Hedland
TPS	Town Planning Scheme
U	
UWMP	Urban Water Management Plan
W	
WA	Western Australia
WAPC	Western Australia Planning Commission

Stage 3 (The Stables)
East Port Hedland

FIGURES

Cardno
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LANDCORP

1:50,000 Scale at A4

Kilometers

0 1

Site Location

FIGURE 1

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Date: 29-01-2015
Coordinate System: GDA 1994 MGA Zone 50
Project: V14018
Map: V14018 FIG01 (Site Location 28.11.14).mxd 01

LANDCORP

Indigenous Heritage Sites

FIGURE 2

Legend

Site Boundary

Indigenous Heritage Sites

N
W E
S

1:10,000 Scale at A4

Metres

0 25 50 100 150 200 250 300 350

Cardno

Map Produced by CARDNO
Date: 29-01-2015
Coordinate System: GDA 1994 MGA Zone 50
Project: V14018
Map: V14018 FIG02 (Indigenous Heritage 5.12.14).mxd 01

LANDCORP

Topography

FIGURE 3

Legend

Site Boundary

Topographic Contours

N
W E
S

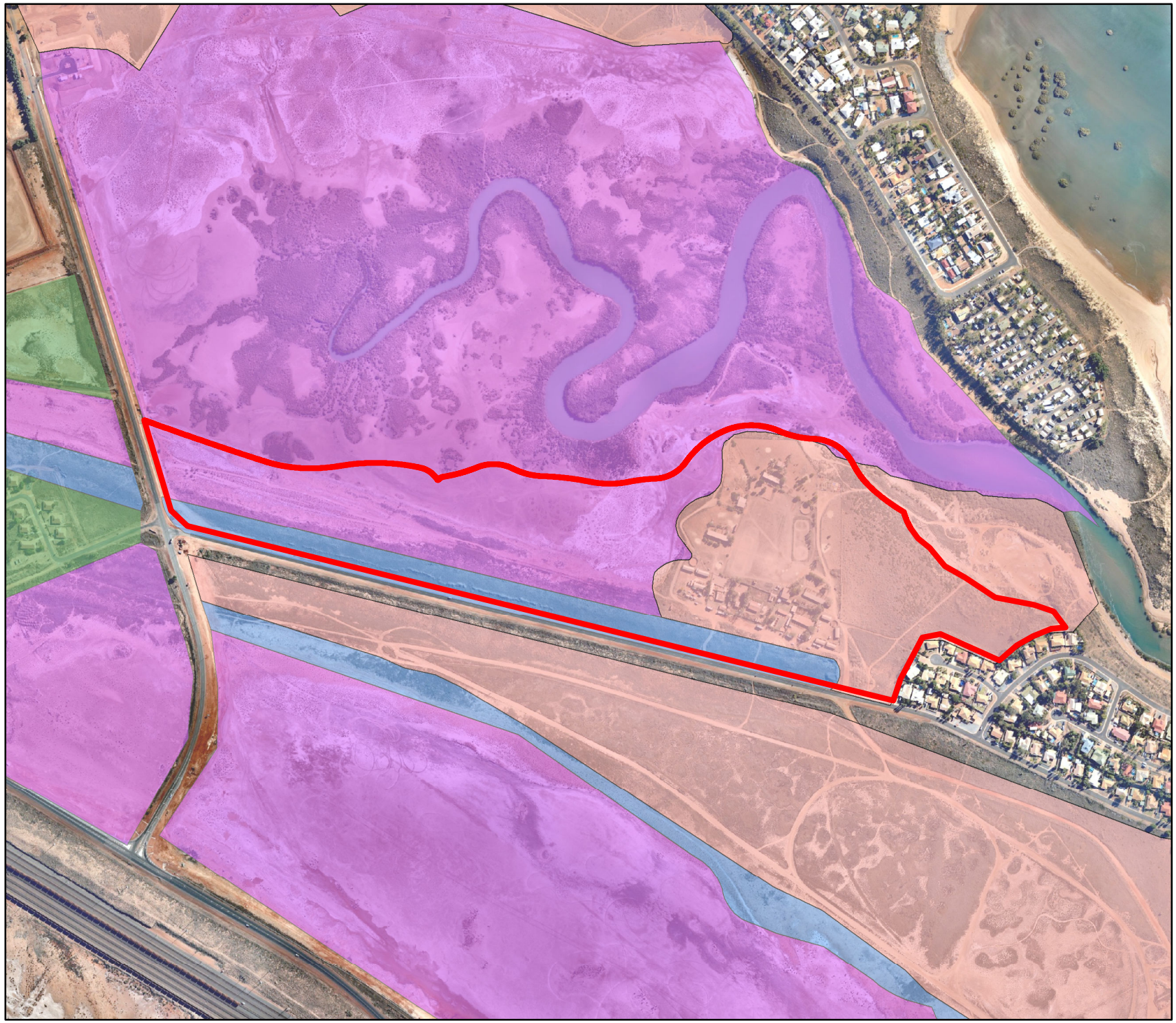
1:8,000 Scale at A4

Metres

0 25 50 100 150 200 250 300

Cardno

Map Produced by CARDNO
Date: 29-01-2015
Coordinate System: GDA 1994 MGA Zone 50
Project: V14018
Map: V14018 FIG03 (Topography 5.12.14).mxd 01



LANDCORP

Surface Geology

FIGURE 4

Legend

- Site Boundary
- Limestone (Calcrete/Calcarenite)
- Sand
- Silty Clay
- Silty Clayey Sand

N
W E
S

Map Reference:
GHD, 2011
Coffey, 2014

1:8,000 Scale at A4

Metres
0 25 50 100 150 200 250 300

Cardno

Map Produced by CARDNO
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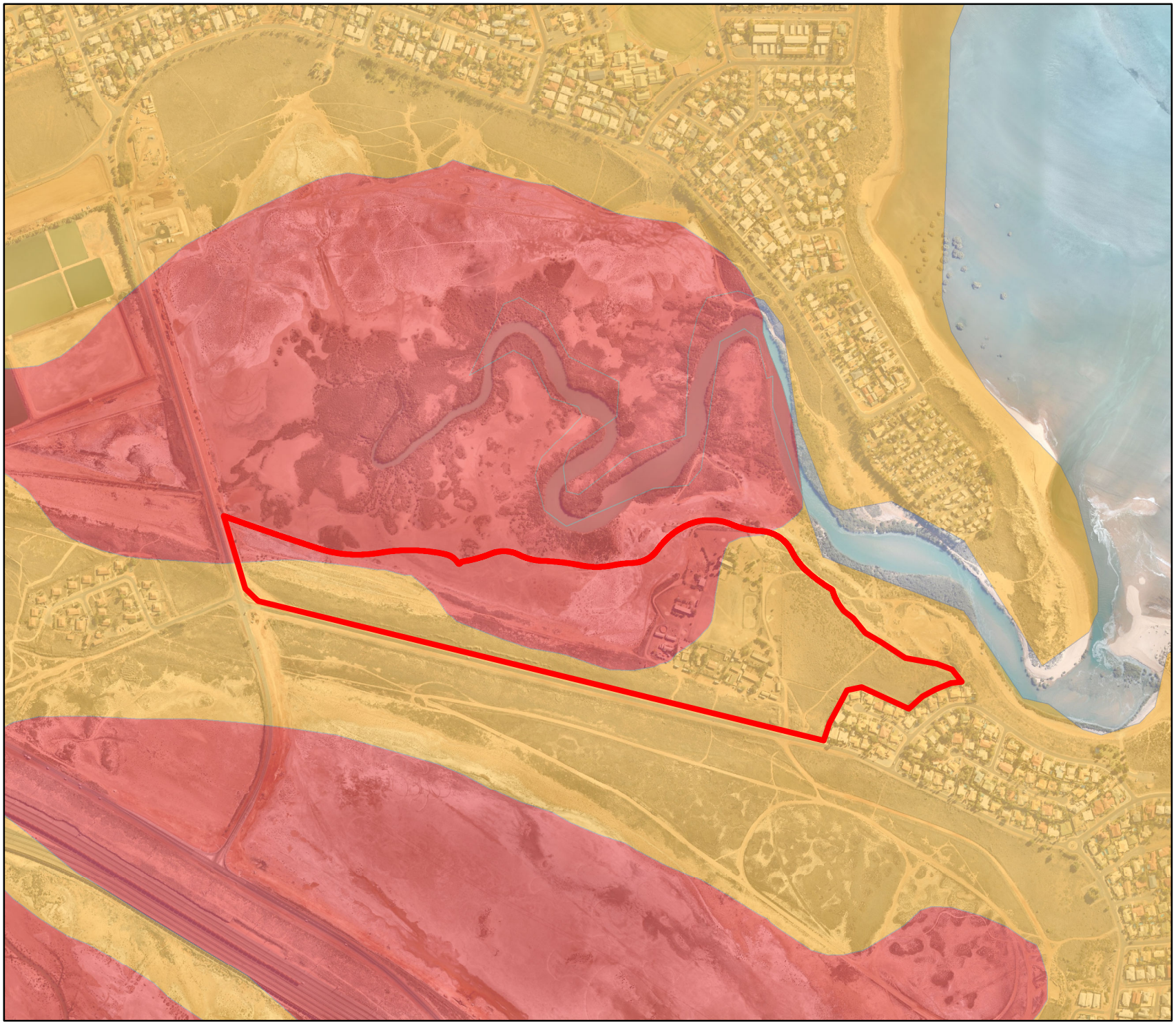
Project: V14018
Map: V14018 FIG04 (Surface Geology 5.12.14).mxd 01



LANDCORP

Acid Sulfate Soils

FIGURE 5

Legend

- Site Boundary
- High to moderate ASS disturbance
- Moderate to low ASS disturbance
- No known ASS disturbance

N
W E
S

1:10,000 Scale at A4

Metres
0 25 50 100 150 200 250 300 350

Cardno

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Coordinate System: GDA 1994 MGA Zone 50
Project: V14018
Map: V14018 FIG05 (Acid Sulfate Soils Mapping 5.12.14).mxd 01

LANDCORP

Pre-Development Catchments and Flow Paths

FIGURE 6

Legend

- Site Boundary
- Flow Paths
- Pre Development Catchments

N
W E
S

1:5,000 Scale at A3

Metres

0 50 100 150 200 250 300

Cardno

Map Produced by CARDNO
Date: 29-01-2015
Coordinate System: GDA 1994 MGA Zone 50
Project:V14018
Map: V14018 FIG06 (Pre Dev Catchments Flow Paths 5.12.14).mxd 01

LANDCORP

Post Development Catchments and Flow Paths of Conceptual Development Option 1

FIGURE 7

Legend

- Site Boundary
- Flow Paths
- Indigenous Heritage Sites
- Trees
- Dual Use Paths
- Swale
- Post Development Catchments
- Verges
- Medium Density Residential (R40)
- Low Density Resdidential (R20)
- Public Open Space (POS)
- Proposed Roads
- Caravan Park

N W E S

1:5,000 Scale at A3

Metres

0 50 100 150 200 250 300

Cardno

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Coordinate System: GDA 1994 MGA Zone 50
Project: V14018
Map: V14018 FIG07 (Post Dev Catchments of Site Concept 01 23.2.15).mxd 01

LANDCORP

Post Development Catchments and Flow Paths of Conceptual Development Option 2

FIGURE 8

Legend

- Site Boundary
- Flow Paths
- Dual Paths
- Trees
- Swale
- Post Development Catchments
- Indigenous Heritage Sites
- Verges
- Proposed Roads
- Medium Density Residential (R40)
- Low Density Residential (R20)
- Public Open Space (POS)
- Caravan Park

N
W E
S

1:5,000 Scale at A3

Metres

0 50 100 150 200 250 300

Cardno

Map Produced by CARDNO
Date: 24-02-2015
Coordinate System: GDA 1994 MGA Zone 50
Project:V14018
Map: V14018 FIG08 (Post Dev Catchments of Site Concept 02 5.12.14).mxd 01

LANDCORP

Conceptual Development Option 1 First 15mm Rainfall Storage

FIGURE 9

Legend

- Site Boundary
- Dual Paths
- Swale
- First 15mm Rainfall
- Trees
- Verges
- Proposed Roads
- Medium Density Residential (R40)
- Low Density Resdidential (R20)
- Indigenous Heritage Sites
- Public Open Space (POS)
- Caravan Park

N W E S

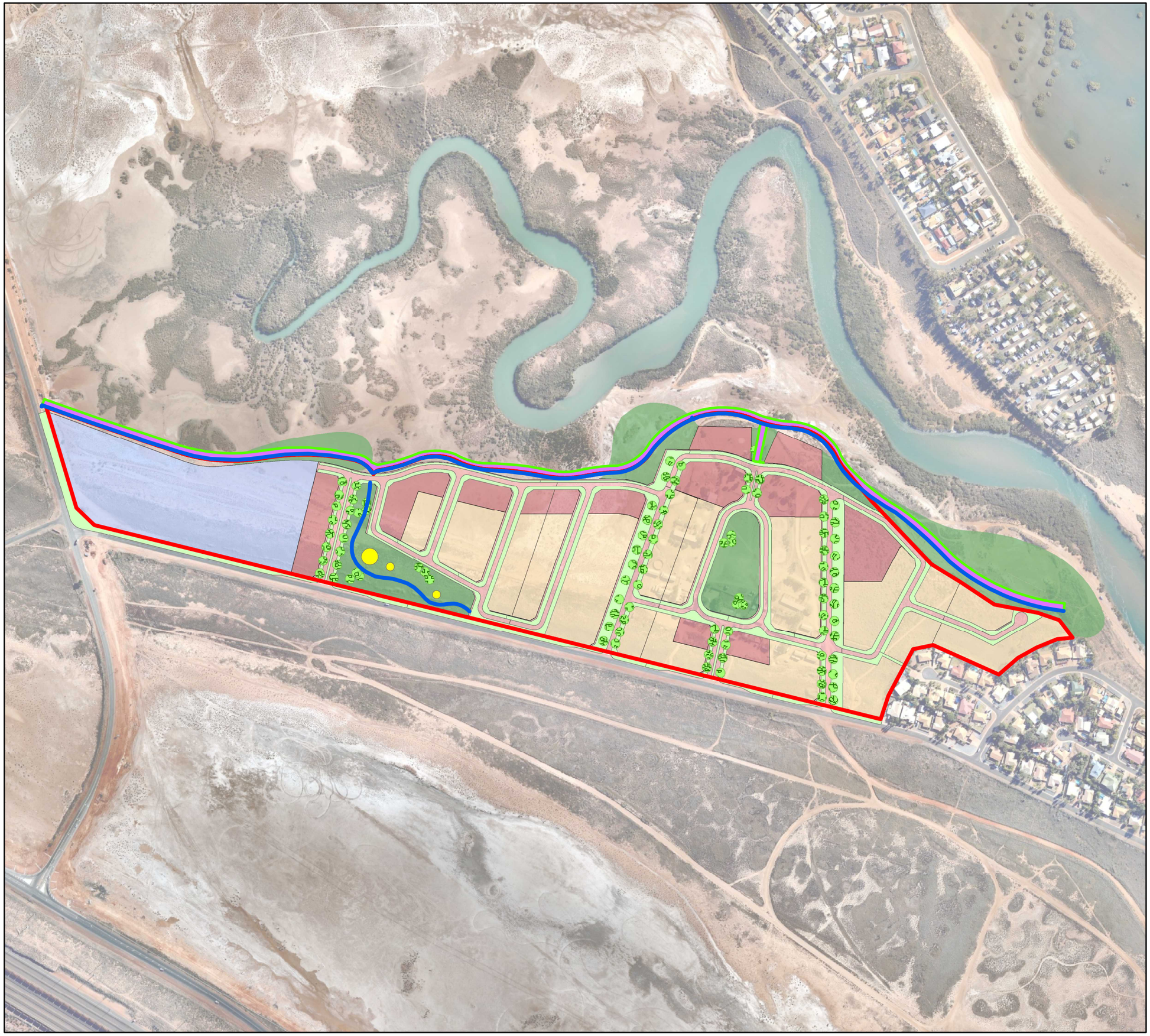
1:5,000 Scale at A3

Metres

0 50 100 150 200 250 300

Cardno

Map Produced by CARDNO
Date: 23-02-2015
Coordinate System: GDA 1994 MGA Zone 50
Project: V14018
Map: V14018 FIG09 (Concept 01 15mm Rainfall Storage 5.12.14).mxd 01



LANDCORP

Conceptual Development Option 2 First 15mm Rainfall Storage

FIGURE 10

Legend

- Dual Paths
- Site Boundary
- Swale
- First 15mm of Rainfall
- Trees
- Verges
- Proposed Roads
- High Density Residential (R40)
- Low Density Residential (R20)
- Indigenous Heritage Sites
- Public Open Space (POS)
- Caravan Park

N
W E
S

1:5,000 Scale at A3

Metres
0 50 100 150 200 250 300

Cardno

Map Produced by CARDNO
Date: 24-02-2015
Coordinate System: GDA 1994 MGA Zone 50
Project:V14018
Map: V14018 FIG10 (Concept 02 15mm Rainfall Storage 5.12.14).mxd 01

Stage 3 (The Stables)
East Port Hedland

APPENDIX A

MANGROVE AND EROSION IMPACTS ASSESSMENT

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Stage 3 (The Stables) East Port Hedland

Mangrove & Potential Erosion Impacts Assessments

V14018

Prepared for
LandCorp

5 February 2015

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Shaping the Future

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RevA	28/11/2014	DRAFT for client	Paul Branson Joanna Garcia-Webb	Justine Jones
RevB	5/2/2015	Footprint assessment	Joanna Garcia-Webb Jo Buckee	Paul Branson Justine Jones

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Figures

1 Introduction

Port Hedland has been identified in the Pilbara Cities Program as an area that should be supported to diversify and grow economically to support a population of 50,000 permanent residents. LandCorp has received funding from the Northern Planning Program to undertake concept planning, and initiate a Scheme Amendment to allow development of Stage 3 (The Stables) East Port Hedland (the site). This is located on the southern side of Pretty Pool Creek, as outlined by the red outline in Figure 1-1.

Figure 1-1 Development locality plan. The Stables (Stage 3) outlined in red (Image source – Nearmap & Google Earth)

Cardno was commissioned by LandCorp to assist them in undertaking background investigations to ensure the site is developable. This will allow a secondary developer to fast track the development process should an opportunity arise.

Cardno's involvement in these background investigations include analysis of:

> Constraints to the developable area due to storm surge

> Potential impacts on the mangroves due to the proposed development

> Potential for erosion of the spit forming the north-western side of the creek mouth

Any impacts to the mangrove system resulting from modification to the Pretty Pool Creek system require investigation as part of State Planning Policy N°. 2.6 - State Coastal Planning Policy (SPP2.6, WAPC, 2013). This includes an assessment of the required setback of the development to allow for coastal processes. In addition, the OEPA and EPA requirements must be met, as stipulated by the EPA Bulletin No. 14 'Guidance for the Assessment of Benthic Primary Producer Habitat Loss in and Around Port Hedland' (EPA 2011).

To comply with SPP2.6, any future development in this region of Port Hedland will be required to be above the 500-year Average Recurrence Interval (ARI) storm tide inundation level for the 100-years planning period

(2110). From the recommendations in the Port Hedland Coastal Vulnerability Study (Cardno 2011), the site would be required to be filled to a minimum level of +6.6 m AHD.

The development site includes a portion of supra-tidal salt flats which are flooded about once a month during the spring tides. Whilst the development footprint does not directly intersect with mangrove vegetation, there is still potential for impacts to mangroves due to the proximity of the development. This would be as a result of alterations of the tidal prism during high water level events and through altered groundwater and surface water regimes.

Hydrodynamic modelling was used to determine any impacts on the mangroves due to decreased tidal prism as a result of changed land levels, with particular reference to:

> Changes to the current velocity through the mangrove area

> Change in mangrove inundation level and duration

> Change in flushing characteristics of the Pretty Pool Creek

The potential for impacts to mangroves from altered groundwater and surface water regimes was investigated using the results of the Local Water Management Strategy (Cardno 2014b).

The eastern portion of the development is located near the mouth of Pretty Pool Creek. It is sheltered from the open ocean by the spit forming the north-western side of the creek mouth. Should the spit be reduced over the planning timeframe, the development may be more susceptible to wave action and erosion processes. In accordance with the risk assessment approach outlined in SPP2.6, an assessment of the risk of erosion of this spit is also documented here.

1.2 Additional Investigations

Cardno completed the above scope of works in November 2014 (Cardno 2014a). LandCorp subsequently requested that additional modelling be undertaken of the Pretty Pool Creek area to minimise the potential impact of the Stables development. In addition, it was recommended to assess the combined potential impact of the proposed Stables development in conjunction with the planned development along Athol Street.

This report assesses the revised footprint, together with the Athol Street footprint. The results herein present the revised Stables development in place of the original footprint presented in Cardno (2014). The revised footprint is presented in Figure 1-2 below. The dashed line shows the original development outline.

Figure 1-2 Development locality plan. The revised Stables (Stage 3) development footprint outlined in red; original footprint is dashed red line. (Image source – Nearmap & Google Earth)

2 Hydrodynamic Modelling

To investigate the effects of changed tidal prism characteristics on the mangroves, Cardno has developed a high resolution Delft3D hydrodynamic model of Pretty Pool Creek that has been coupled to the large scale, calibrated storm tide Delft3D model utilised in the Port Hedland Coastal Vulnerability Study (PHCVS) (Cardno, 2011). The tidal processes were simulated for the existing bathymetry, the revised Stables developed layout, and the revised Stables layout combined with the Athol Street footprint. This enabled differences in exchange and velocity to be investigated.

The following scenarios were simulated for each of the existing, Stables and Stables / Athol combined development layouts:-

> Scenario 1: 4-week scenario under ambient conditions. This captured the full spring-neap tidal cycle, enabling an understanding of the extent of the inundation of the mangroves under ambient conditions.

> Scenario 2: 2-years ARI storm event. Changes to the more frequently occurring conditions could impact the mangroves more than the large storms. As such, simulating the 2 year ARI event provides a better understanding of these impacts.

> Scenario 3: 20-year ARI storm event. This storm event is more likely to occur during the design life of the development than Scenario 4 below, so provides information from which to assess potential impacts on the mangroves during extreme conditions.

> Scenario 4: 500-years ARI simulation. This event is selected to be in line with the SPP2.6 storm event for erosion for this region of Western Australia. A cyclonic event was selected from a database of 16,000 synthetic cyclone tracks (Cardno, 2011) that is equivalent to a 500-years ARI storm tide event near this site. In line with the planning policy, this is coupled with the recommended 0.9 m sea-level rise.

2.1 Model Setup

The model grid and bathymetry for the Port Hedland Town and Pretty Pool Creek areas is presented in Figure 2-2 and Figure 2-2 below. A high resolution grid through the Pretty Pool Creek and adjacent township was developed. For display purposes only every second grid line is shown for the Town grid and every third grid line for the Pretty Pool grid. Throughout the narrow creek region, the model has a resolution of approximately 5 m, providing approximately five grid points across the creek which reproduces the creek bed bathymetry well. This finer grid over the creek is omitted from Figure 2-1 for clarity.

A multi-beam hydrographic survey was carried out over Pretty Pool Creek for a previous study undertaken by Cardno for LandCorp (Surrich, 2012). This is incorporated into Cardno's Digital Elevation Model (DEM) of the Port Hedland region, which includes high resolution LiDAR data of the surrounding land regions. In addition LandCorp provided LiDAR data from 2014, and this data was utilised to update the bathymetry of the Pretty Pool Creek entrance and nearshore tidal flats.

Due to the large, flat mangrove and salt-marsh area surrounding Pretty Pool, which becomes inundated under extreme water levels, the 'Flooding' momentum advection scheme has been utilised (Stelling and Duinmeijer, 2003). This scheme has been specifically developed for the inundation of dry land with obstacles involved, such as roads.

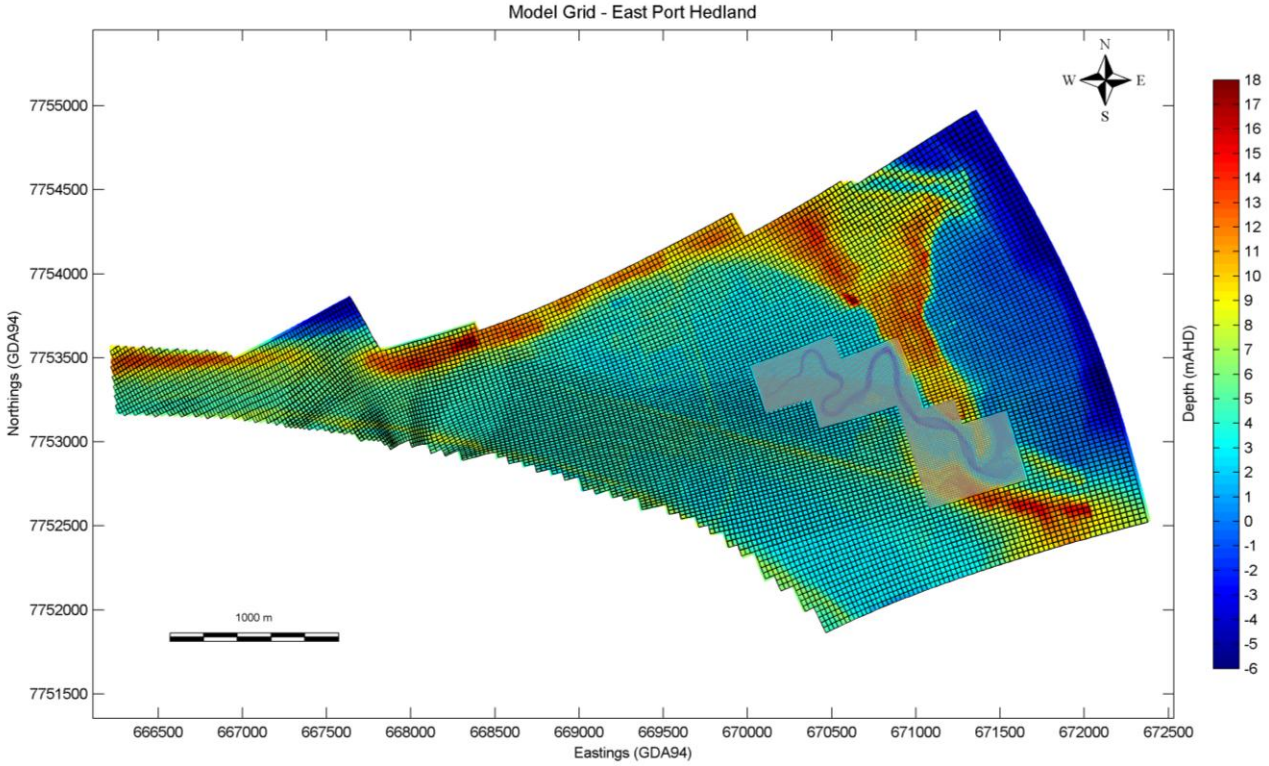

Figure 2-1 Model grid and bathymetry of Inner Port Hedland Town. Note that only every 2nd gridline is displayed

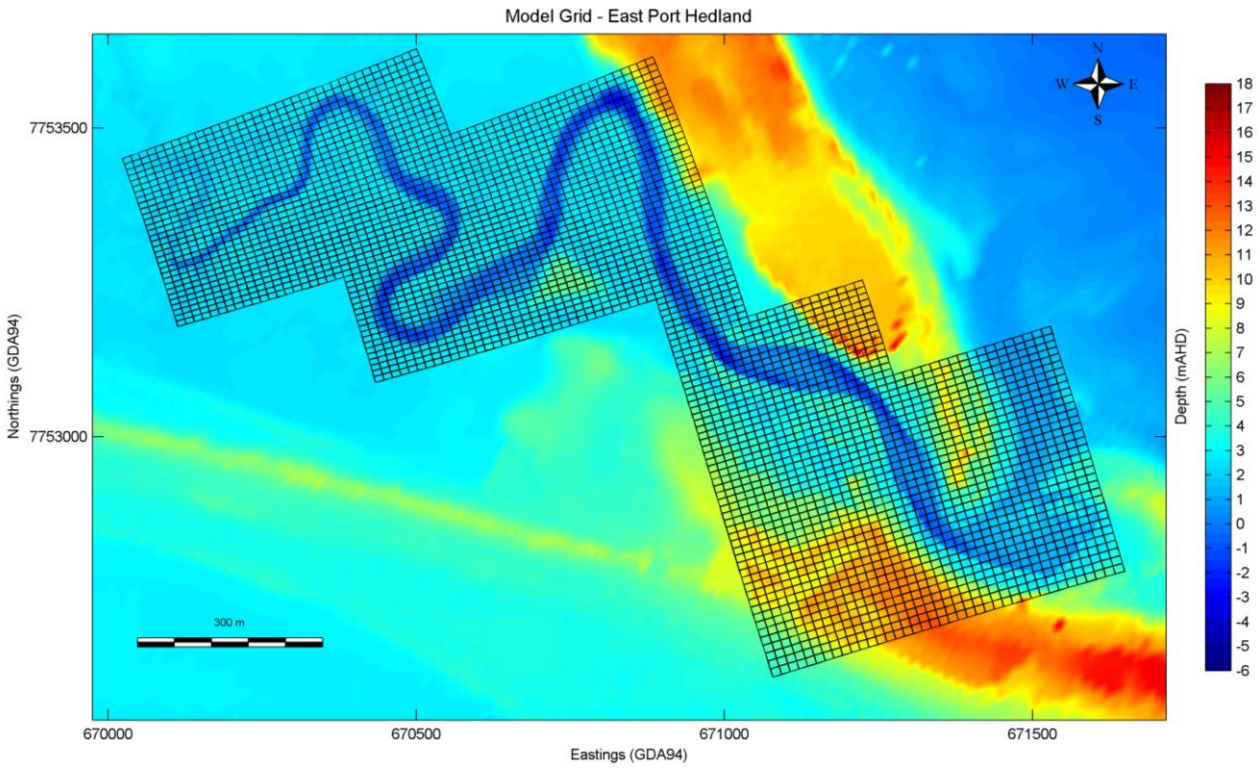

Figure 2-2 Model grid and bathymetry of Pretty Pool Creek area. Note that only every 3rd gridline is displayed

2.2 Model Calibration and Hydraulic Roughness

A spatial roughness map was implemented throughout the Pretty Pool Creek region and surrounding East Port Hedland township that utilised the Manning's 'n' roughness scheme. The prescribed hydraulic roughness was based on digitising different land use and vegetation zones. These are consistent with the overland flow model utilised in the PHCVS (Cardno, 2011). In addition, a specific calibration of the creek bed roughness was undertaken. The mangrove regions and creek bed were prescribed with a Manning's 'n' value of 0.18 and 0.02, respectively. Figure 2-3 presents a plot of the measured and modelled water level at two locations within the creek. Very good agreement between modelled and measured water levels is evident.

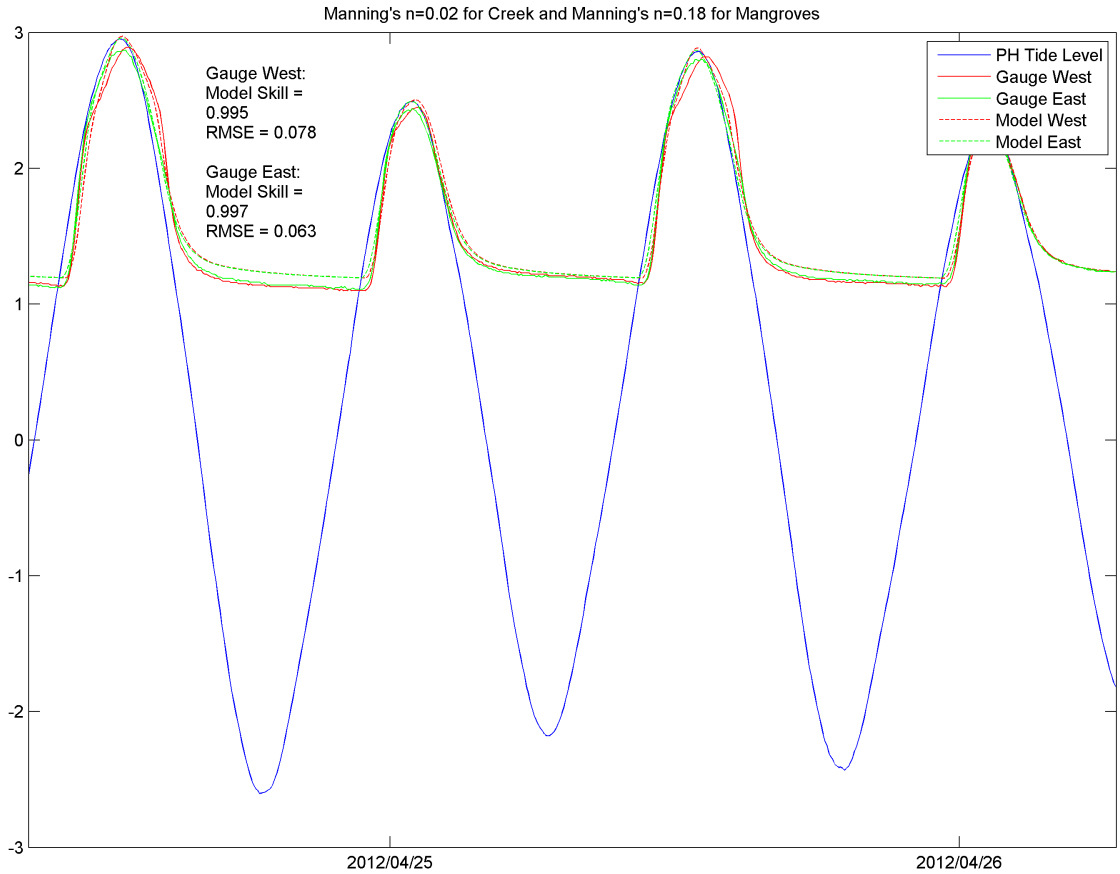

Figure 2-3 Model validation against measured water level data at two gauging locations within Pretty Pool Creek

2.3 Development Footprint Refinement

To select an optimised development footprint, the 500-years ARI scenario was simulated for 3 layout adjustments: Options 1 to 3. These layouts are presented in Figure 2-4. The options aimed at increasing the channel width near the creek mouth, so as to minimise the change to flow conditions. This is the area showing the most change to conditions for the original development footprint (Cardno 2014a).

The difference in inundation depth and current speed between the three options and the existing (base) case are presented in Figure 2-5 and Figure 2-6 respectively. Each option showed improvement compared to the original development footprint (as reported in Cardno 2014a). As negligible differences were observed between Options 2 and 3, Option 2 was considered optimal for the channel width, and selected as the revised development footprint for further analysis. This is the footprint displayed in Figure 1-2 in Section 1.2.

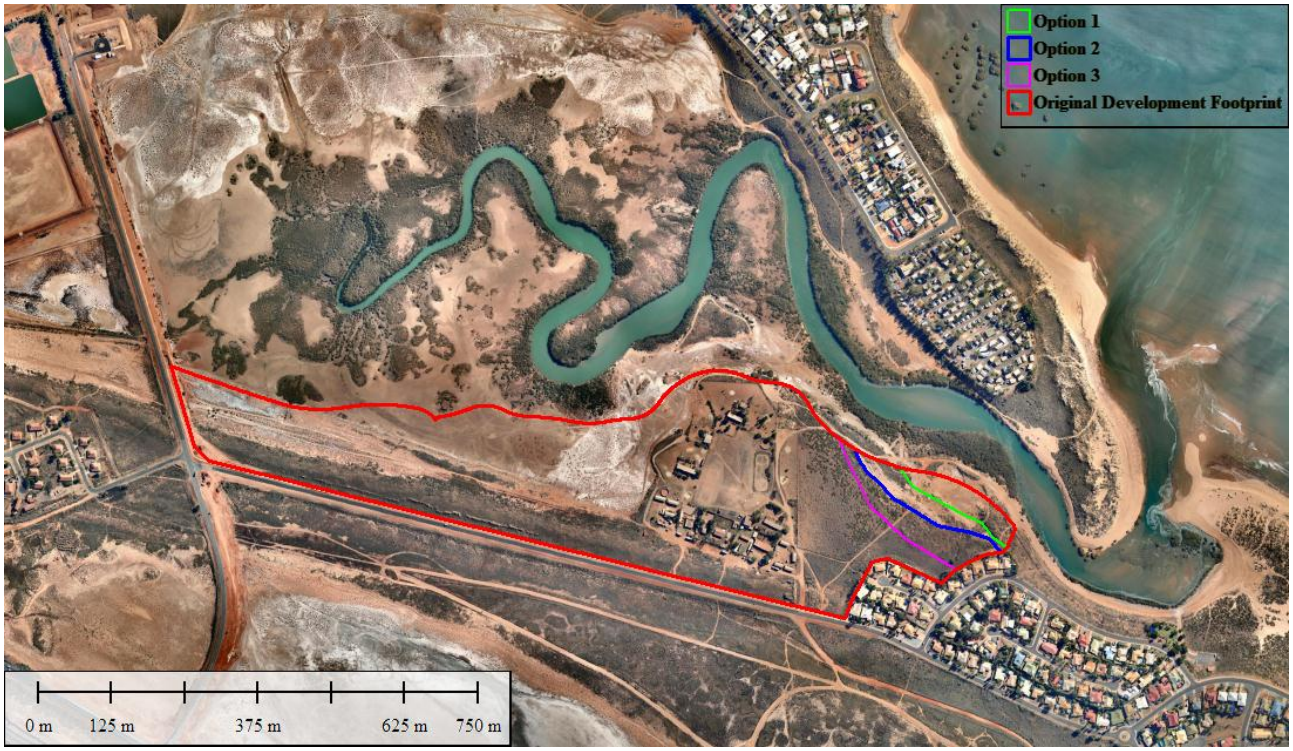

Figure 2-4 Development Options 1 to 3, and original footprint

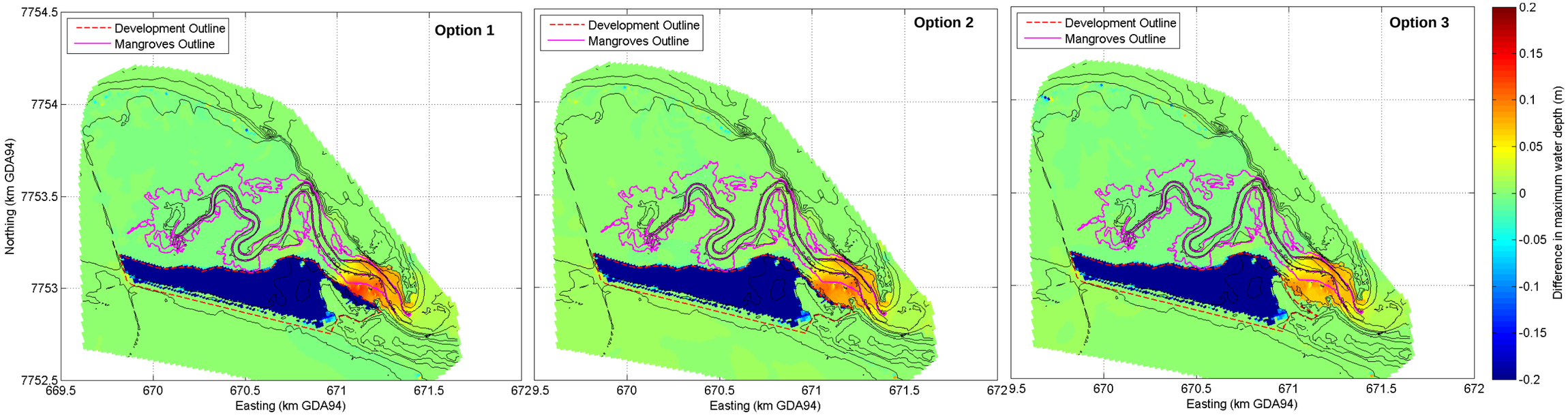

Figure 2-5 Differences in maximum inundation depth for 500-years ARI storm conditions. Positive change indicates that the depth is greater for the Design Case. The dark blue areas indicate regions that are no longer inundated

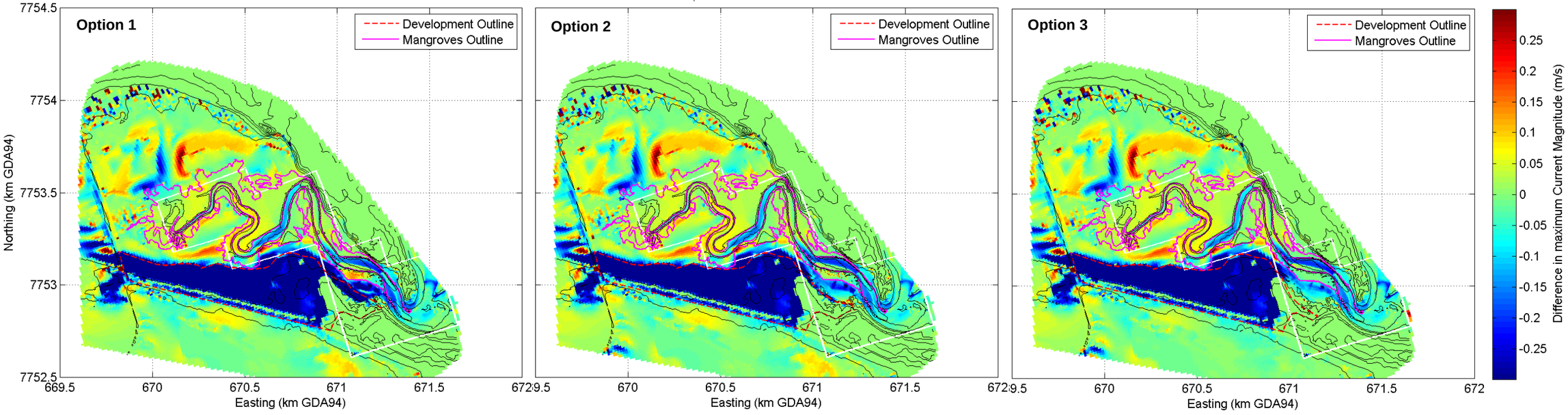

Figure 2-6 Differences in maximum current magnitude for 500-years ARI storm conditions. Positive change indicates that the current magnitude is greater for the Design Case. White outline is the boundary between domains

2.4 Model Scenarios

Four forcing scenarios were modelled:

> Scenario 1: An ambient 1-month simulation that encompassed two spring tides with a mean high water level equal to approximately MHWS (2.8 m AHD). This is a typical tidal scenario.

> Scenario 2: A 7-day simulation with for a 2-years ARI spring tide level of approximately 3.3 m AHD

> Scenario 3: An extreme 20-years ARI tropical cyclone event (TC Kerry, Jan 1973)

> Scenario 4: An extreme 500-years ARI tropical cyclone event, incorporating 0.9 m sea-level rise (SLR).

2.4.1 Development Cases

A design fill level of +6.6 m AHD across the proposed development areas was incorporated into the model to assess the influence of the development layouts on the mangrove hydrodynamics; refer to Figure 2-7 to Figure 2-9 for the existing, Stables, and Stables combined with Athol Street design bathymetry respectively. All layouts were modelled with the forcing scenarios described above. For ease of reference, the following labels are used herein to refer to each of the cases:

> Base case – no development

> Design Case - Stables only development

> Athol Design Case - Stables combined with Athol Street development.

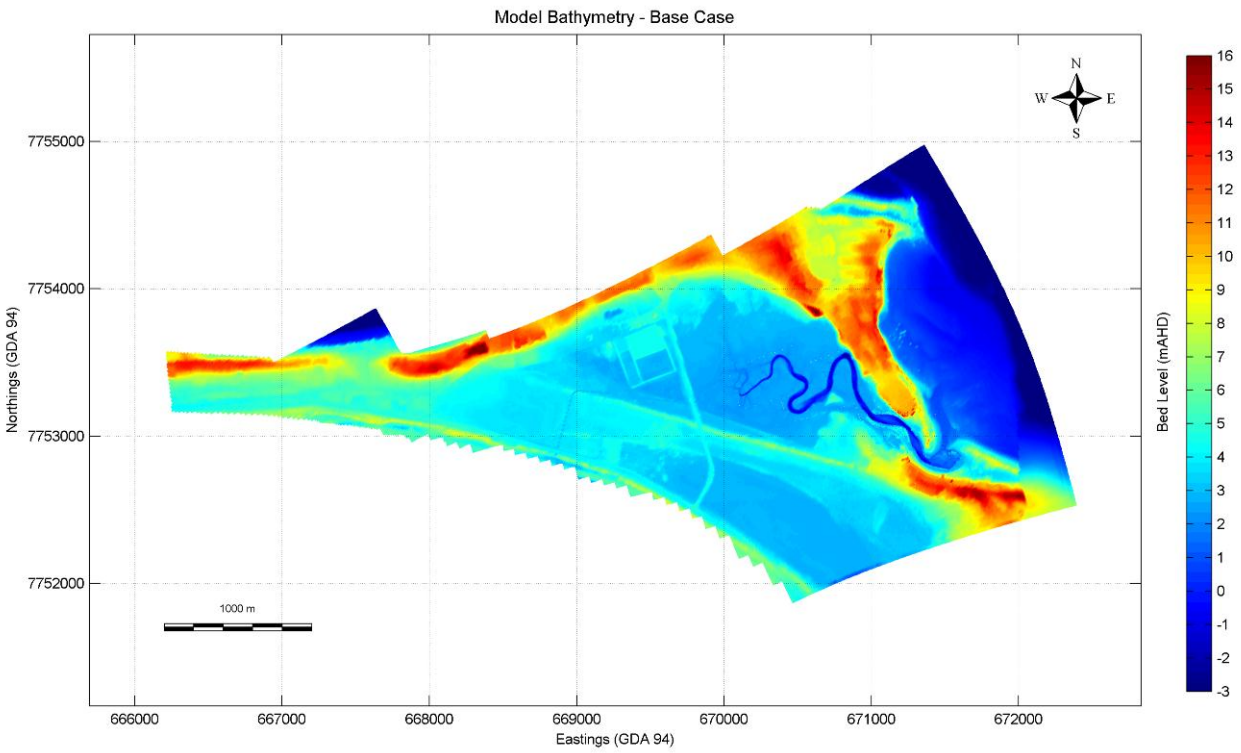

Figure 2-7 Base Case bathymetry of the East Port Hedland area

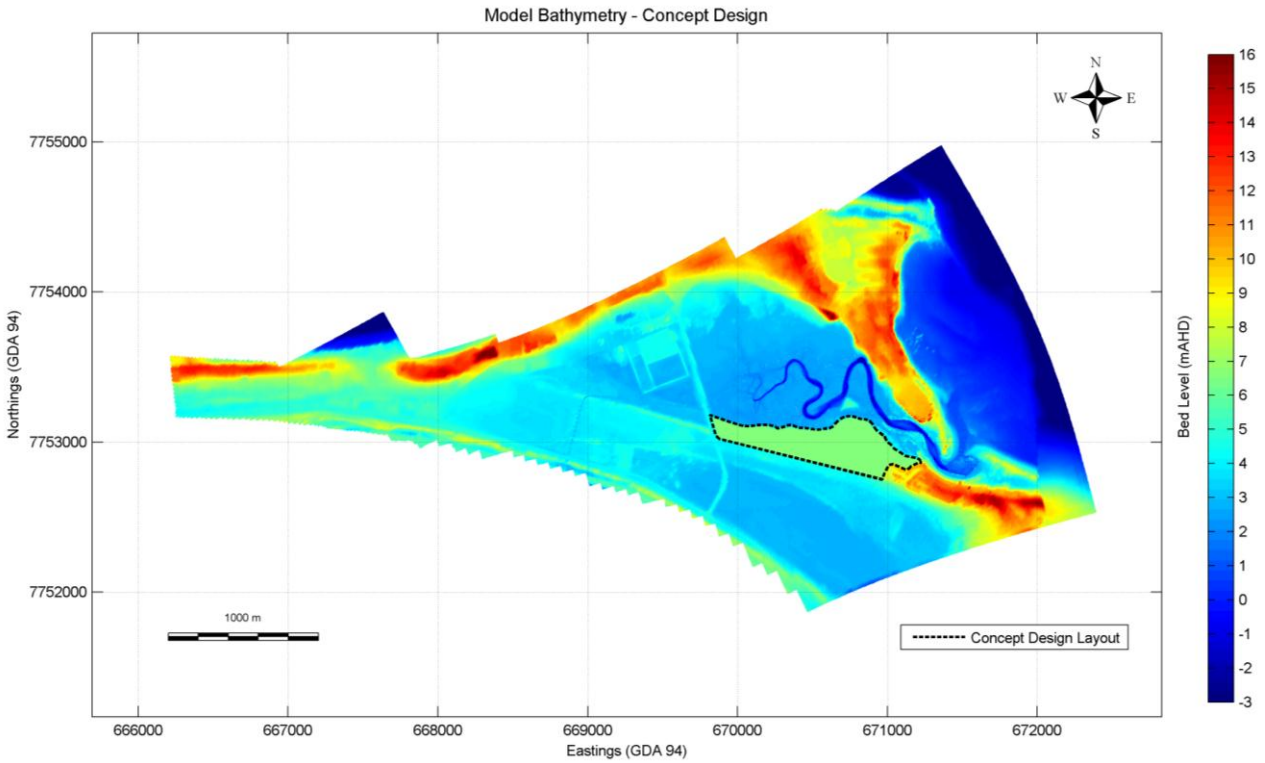

Figure 2-8 Design Case bathymetry of the East Port Hedland area, including the proposed Stables development Design Outline

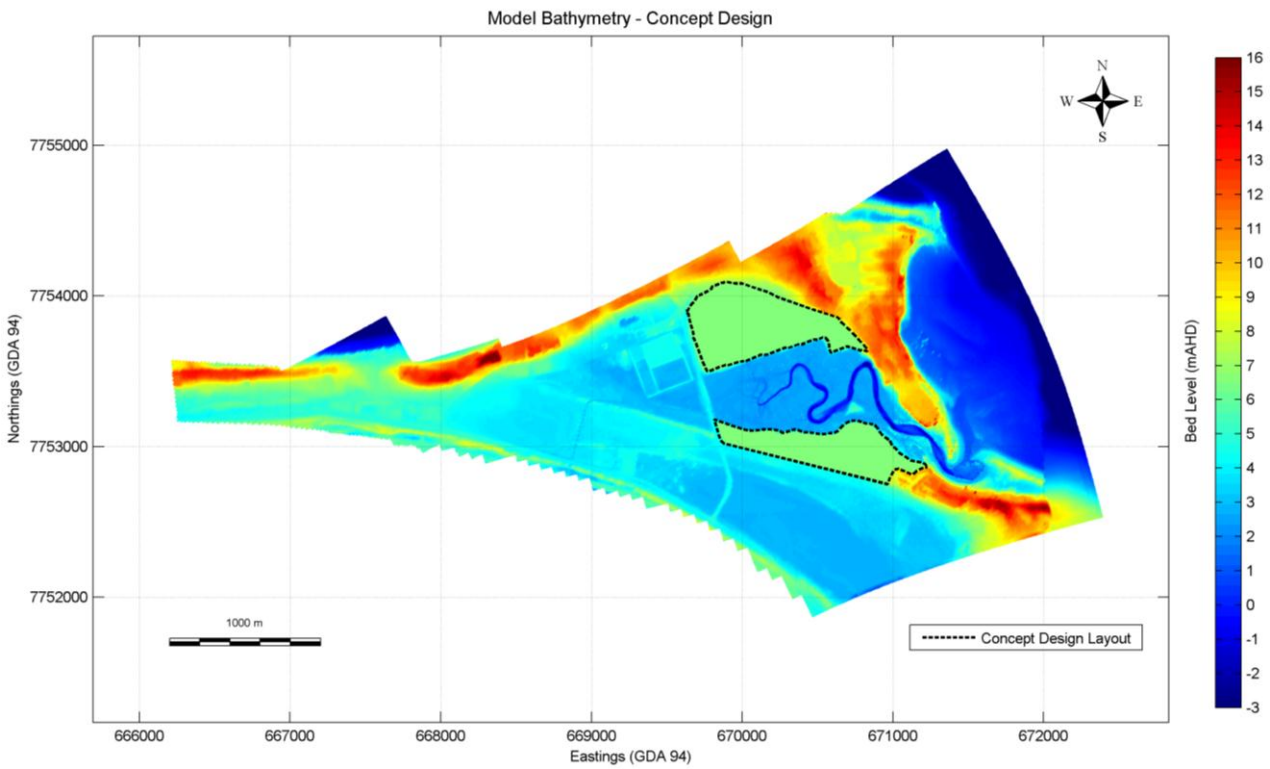

Figure 2-9 Athol Design Case bathymetry of the East Port Hedland area, including both the proposed Stables and Athol Street development Design Outlines

2.5 Model Results

The change in discharge through a cross section at the entrance to Pretty Pool Creek due to the Stables development is shown in Table 2-1. For each simulation, the total discharge was calculated across this cross-section for both the flood and ebb tides. The mean of the flood and ebb magnitudes was taken and is displayed in the table.

Table 2-1 Difference in discharge through the Pretty Pool Creek entrance.

Scenario	Discharge Through Cross-Section Over Simulation (Mean of Flood & Ebb Magnitudes)			Difference Design (%)	Difference Athol Design (%)
	Existing (m^3)	Design (m^3)	Athol Design		
1	5.08×10^6	5.05×10^6	5.05×10^6	-0.4	-0.5
2	1.80×10^6	1.80×10^6	1.79×10^6	-0.2	-0.3
3	3.75×10^6	3.63×10^6	3.39×10^6	-3.1	-9.5
4	1.24×10^7	1.18×10^7	1.04×10^7	-5.4	-16.1

These results indicate that there is a very minor reduction in the volume of water flowing into and out of the creek across a 1-month tidal cycle (Scenario 1) and a 2-year ARI tidal condition (Scenario 2). This suggests there is little modification to the hydrodynamic regime of the Pretty Pool Creek under Scenarios 1 and 2.

For Scenario 3, the 20-years ARI storm event, there is a 3% reduction in the volume of water that flows into and out of the creek as a result of the Stables Development. With the addition of the Athol Street footprint, there is a 9% reduction.

For Scenario 4, the 500-years ARI storm event, there is a 5% reduction in the volume of water that flows into and out of the creek as a result of the Stables Development. With the addition of the Athol Street footprint, there is a 16% reduction.

The development layouts reduce the area available to be inundated under high water level. This in turn reduces the volume flux and associated frictional losses, allowing the water level within the creek region and surrounds to reach a higher quasi-equilibrium with the oceanic water level. Note for the Stables footprint by itself, these reductions are less than that predicted from just the Athol Street Development (Scenarios 3 and 4 predicted 6% and 11% reductions respectively (Cardno, 2012)).

Water levels are increased by approximately 3-7 cm across parts of the mangrove regions for the extreme events due to the Stables Development, and 5 - 20 cm for the Athol Design Case (refer Sections 2.5.4 and 2.5.5 below).

2.5.2 Scenario 1: Ambient/monthly event

Maximum water levels and current magnitudes for the pre and post-development cases are mostly comparable across the mangrove region for the typical tidal scenario (Figure 2-10 and Figure 2-11).

There is a minor increase in the extent of inundation of the salt flats, with negligible changes within the creek and mangrove areas under ambient conditions. Some areas of the salt flats that were previously inundated are no longer due to the presence of the development layout (shown in dark blue).

For the Athol Design Case, these is an increase in water depth throughout the area of approxiomately 3 cm due to the reduced area available for inundation. The currents are similar for both the Design Case and the Athol Design Case.

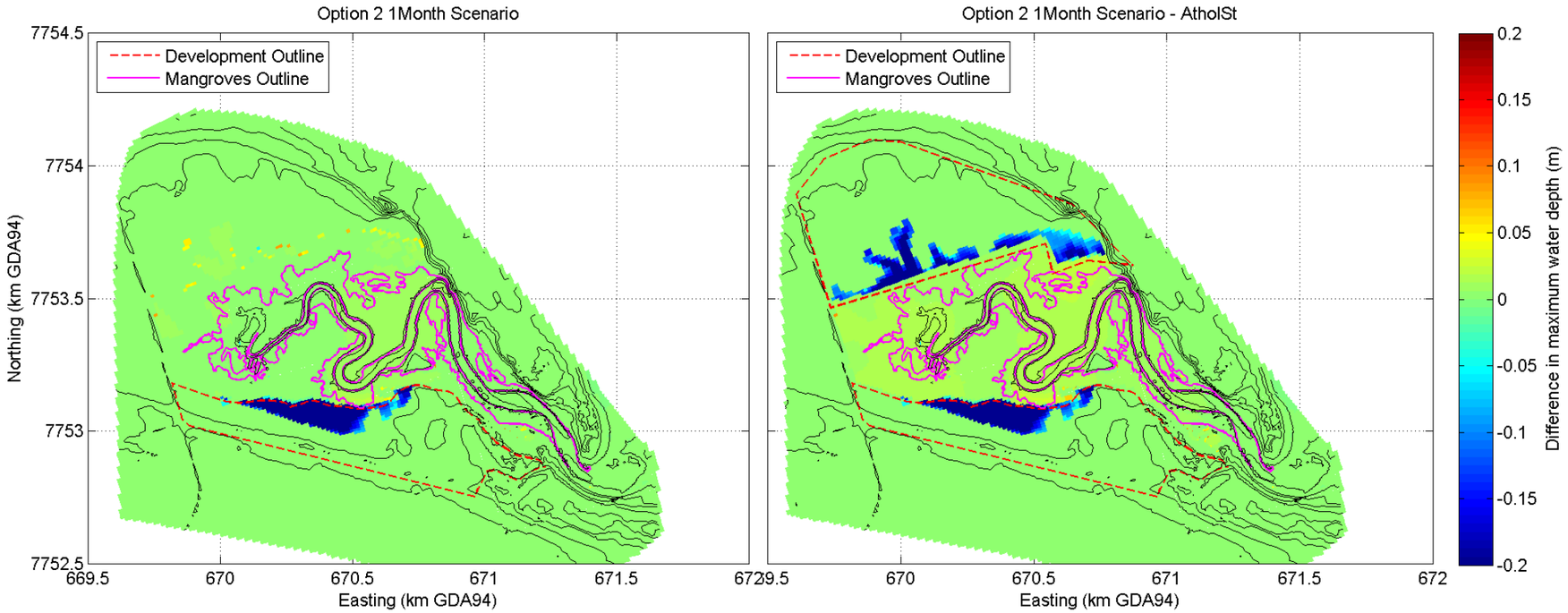

Figure 2-10 Differences in maximum inundation depth for Scenario 1 - Design and Athol Design Cases. Positive change indicates depth greater for Design Cases. Dark blue areas indicate regions no longer inundated

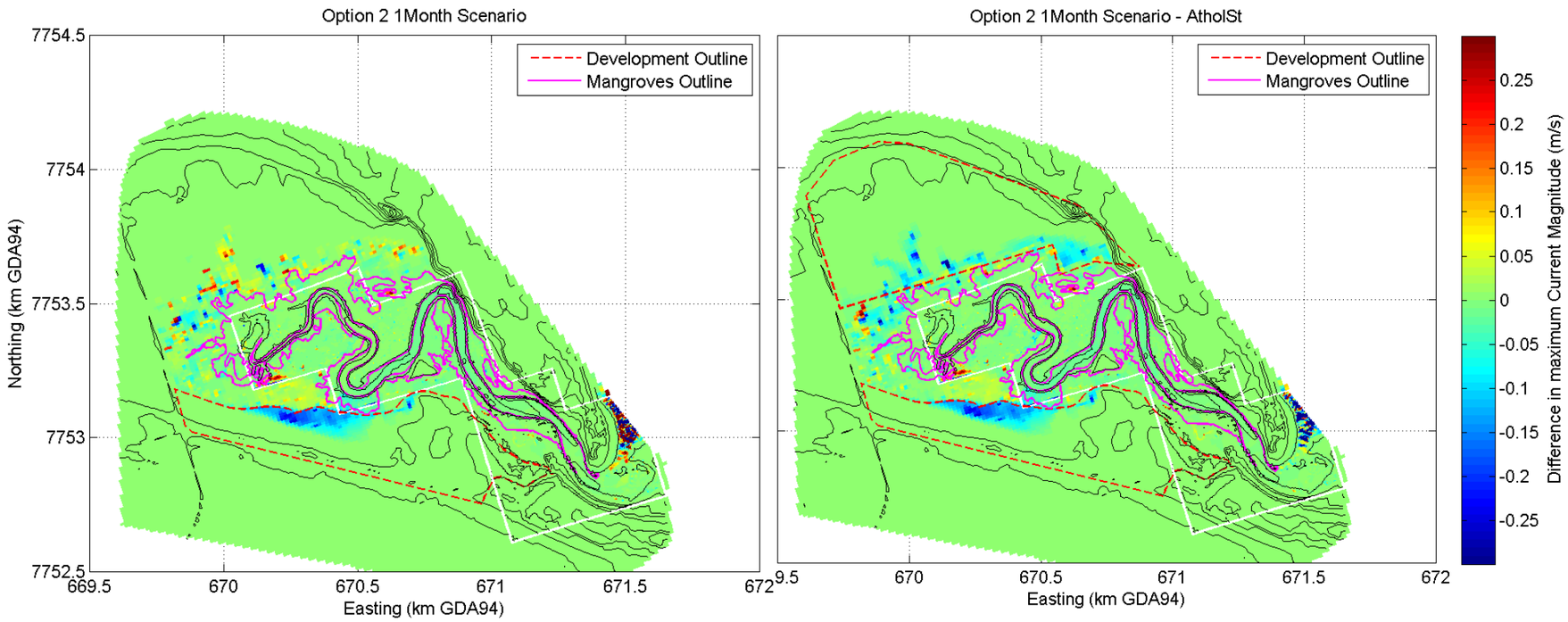

Figure 2-11 Differences in maximum current magnitude for Scenario 1 - Design and Athol Design Cases. Positive change indicates current magnitude greater for Design Cases. White outline is boundary between domains

2.5.3 Scenario 2: 2-year ARI Event

Maximum water levels and current magnitudes for the pre and post-Stables development cases are mostly comparable across the mangrove region for the 2-years ARI high tide scenario (Figure 2-12 and Figure 2-13).

There is a minor increase in the extent of inundation of the salt flats, with negligible changes within the creek and mangrove areas under ambient conditions. Some areas of the salt flats that were previously inundated are no longer due to the presence of the development layout (shown in dark blue).

There is an increase in current magnitude in the tidal flats of approximately 5 cm/s adjacent to the development to the west (indicated by the arrow in Figure 2-13).

For the Athol Design Case, these is an increase in water depth throughout the area of approxiomately 4 cm due to the reduced area available for inundation. The currents are similar for both the Design Case and the Athol Design Case.

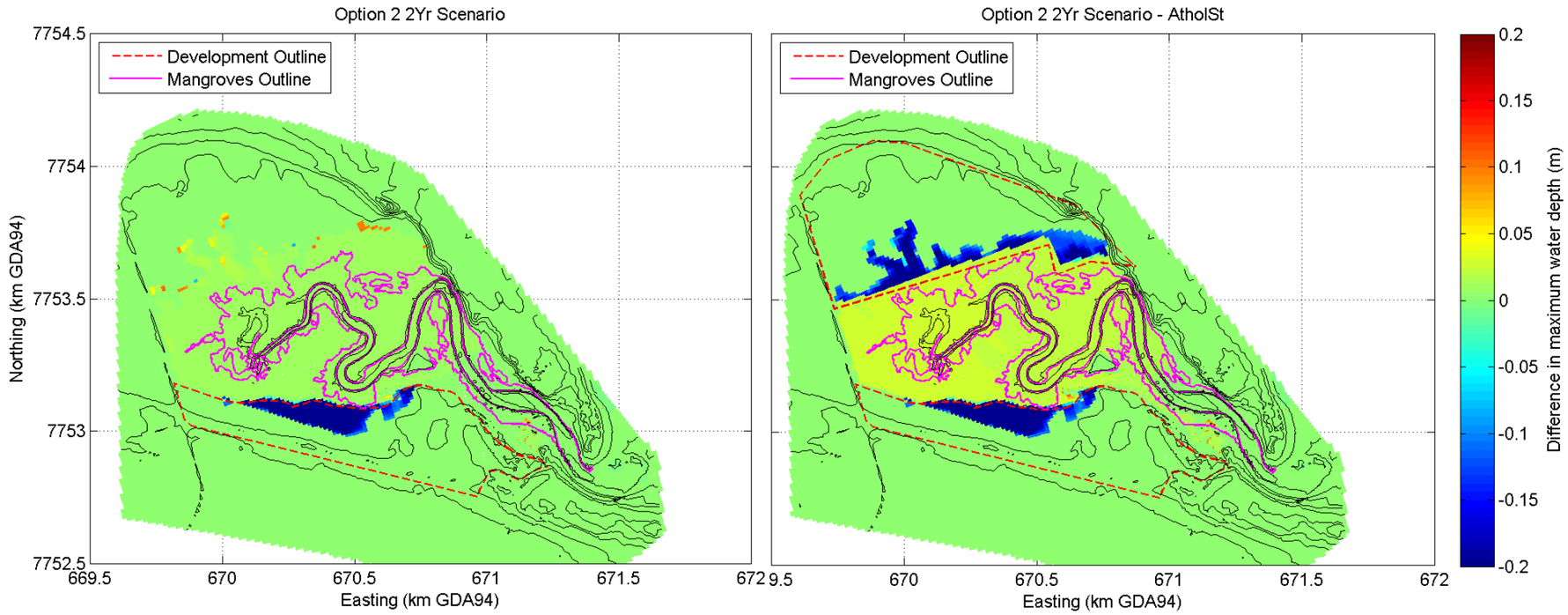

Figure 2-12 Differences in maximum inundation depth for Scenario 2 - Design and Athol Design Cases. Positive change indicates depth is greater for Design Cases

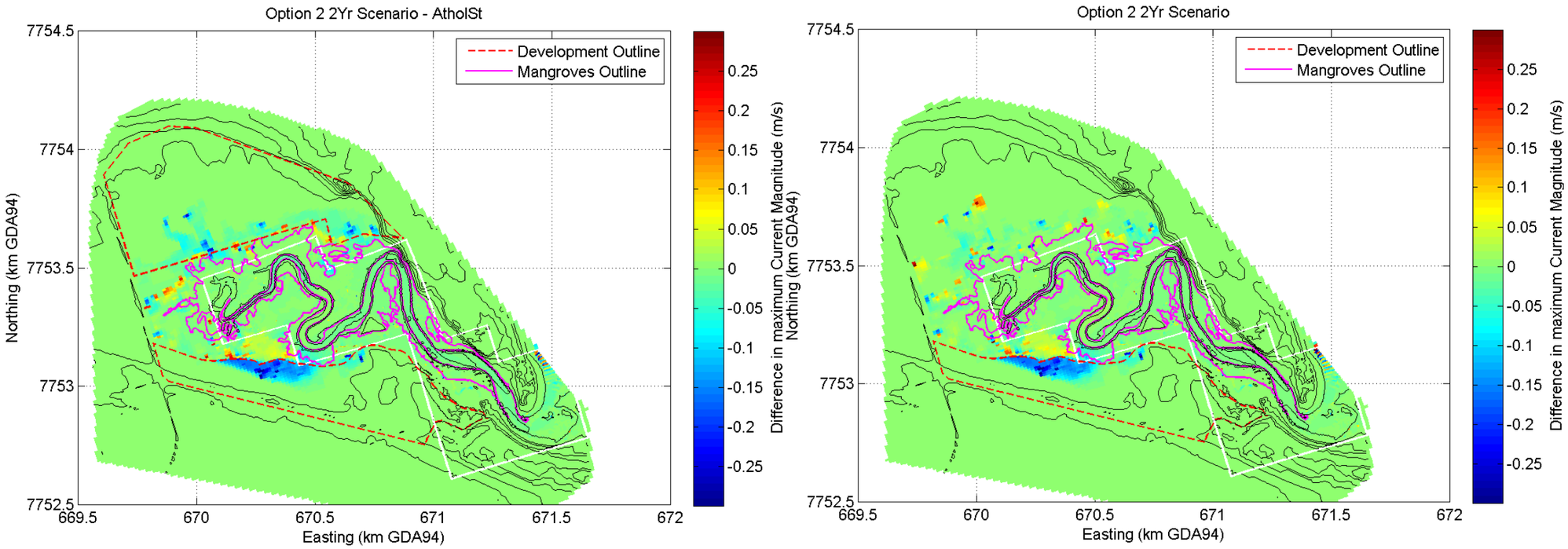

Figure 2-13 Differences in maximum current magnitude for Scenario 2 - Design and Athol Design Cases. Positive change indicates current magnitude is greater for Design Cases. White outline is boundary between domains

2.5.4 Scenario 3: 20-year ARI Event

The difference plots of the modelled maximum inundation depth, Figure 2-14, shows water depths are predicted to be approximately 3-5 cm higher across the Pretty Pool region during a 20-years ARI storm due to the Design Case alone. The Athol Design case shows increases of up to 7 cm. This is a small increase compared to the inundation depth of 2-2.5 m shown in Figure 2-15; roughly a 1-3% increase.

The difference in current magnitudes over the duration of the storm for both the existing and design cases and the maximum current magnitudes are plotted in Figure 2-16 and Figure 2-17 respectively. The maximum current magnitude through the majority of the mangroves is approximately 30-60 cm/s for the existing layout. Current speeds in the channels near the entrance exceed 100 cm/s.

The presence of the Design Case development slightly reduces the width of a flow path on the southern side of the creek near the entrance (indicated by the arrow in Figure 2-16). This has in turn increased the current magnitude through this section by approximately 7 cm/s. The high variability in the results in this area is due to the variations in the flow paths and peak currents associated with the modelled wetting and drying process (indicated by the arrow in Figure 2-17).

The addition of the Athol St footprint leads to increases in currents at the development footprint boundary of up to 20 cm/s.

Assuming (a smooth) Nikuradse equivalent sand grain roughness for the mangrove sediments of approximately 1 mm; suspension of fine material (with a nominal critical shear stress for mobilisation between 0.2 and 0.6 N/m^2) occurs for current magnitudes that exceed approximately 40 to 60 cm/s, at flow depths of approximately 2 m. The Base Case model results indicate that these conditions are present for the 20-years ARI. An increase of 7 cm/s and 20 cm/s due to the Design Case and Athol Design Case respectively will result in slight alteration to the sediment redistribution that occurs during these extreme events. This may result in slight redistribution of the mangrove habitat, however given the frequency of these events it is likely that other direct impacts (i.e. wind) will likely be more significant than the alterations to the hydrodynamic regime due to the development.

Due to the predicted increased water levels within the proximity of Pretty Pool Creek resulting from the Design and Athol Design Case developments, alterations to the inundation of adjacent areas can occur (Figure 2-18). Overtopping of Cook Point Drive is predicted to occur in the 20-year ARI event leading to approximately 20 cm of inundation over an increased area to the west of Cook Point Drive compared to the existing layout for both the Design Cases. The inclusion of the Athol Street development (Athol Design Case) results in a significantly broader area being inundated compared to the Design Case alone. The corresponding change to current magnitude is also included in Figure 2-19 for this zoomed-out view.

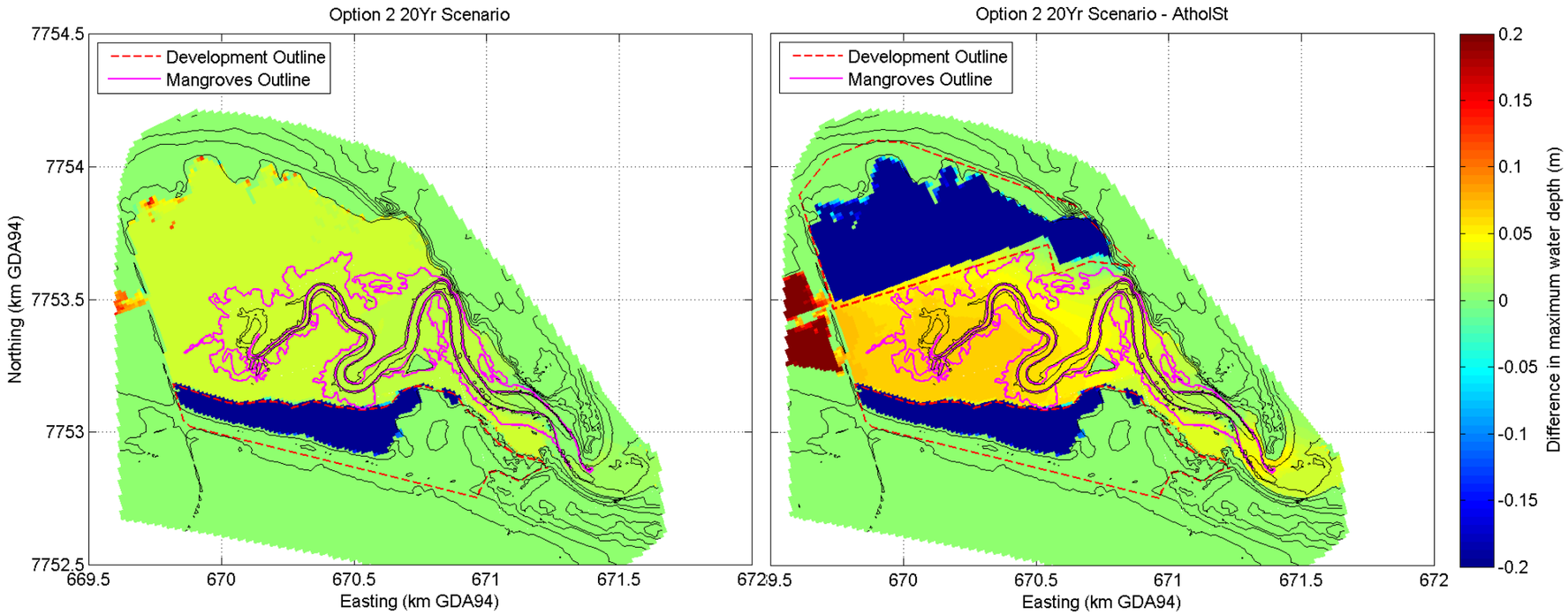

Figure 2-14 Differences in maximum inundation depth for Scenario 3 - Design and Athol Design Cases. Positive change indicates depth is greater for the Design Cases

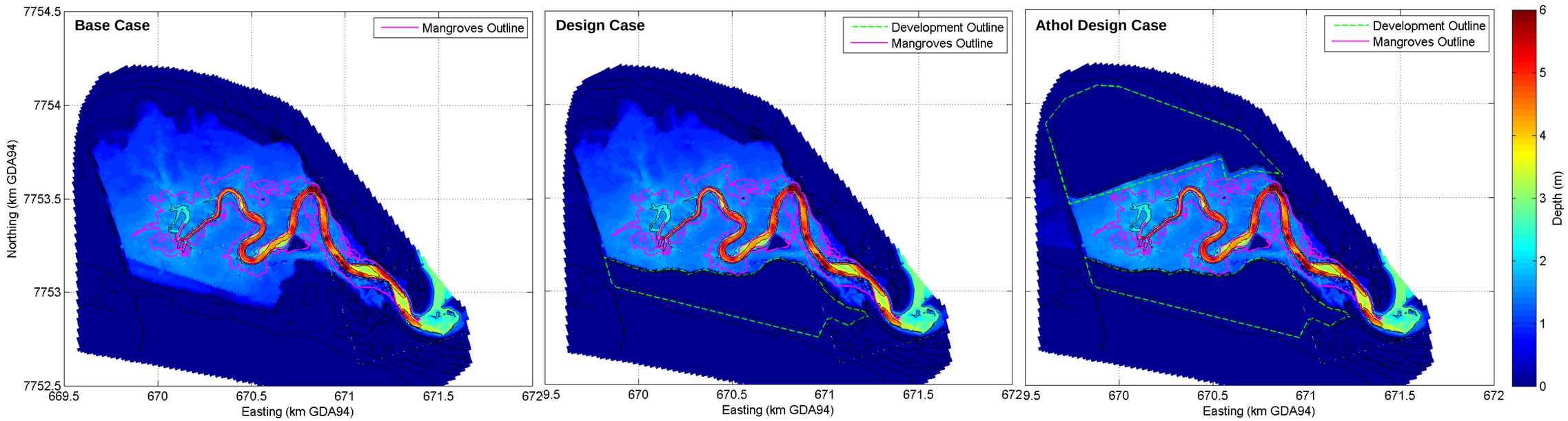

Figure 2-15 Maximum water depth for Base, Design and Athol Design Cases for Scenario 3

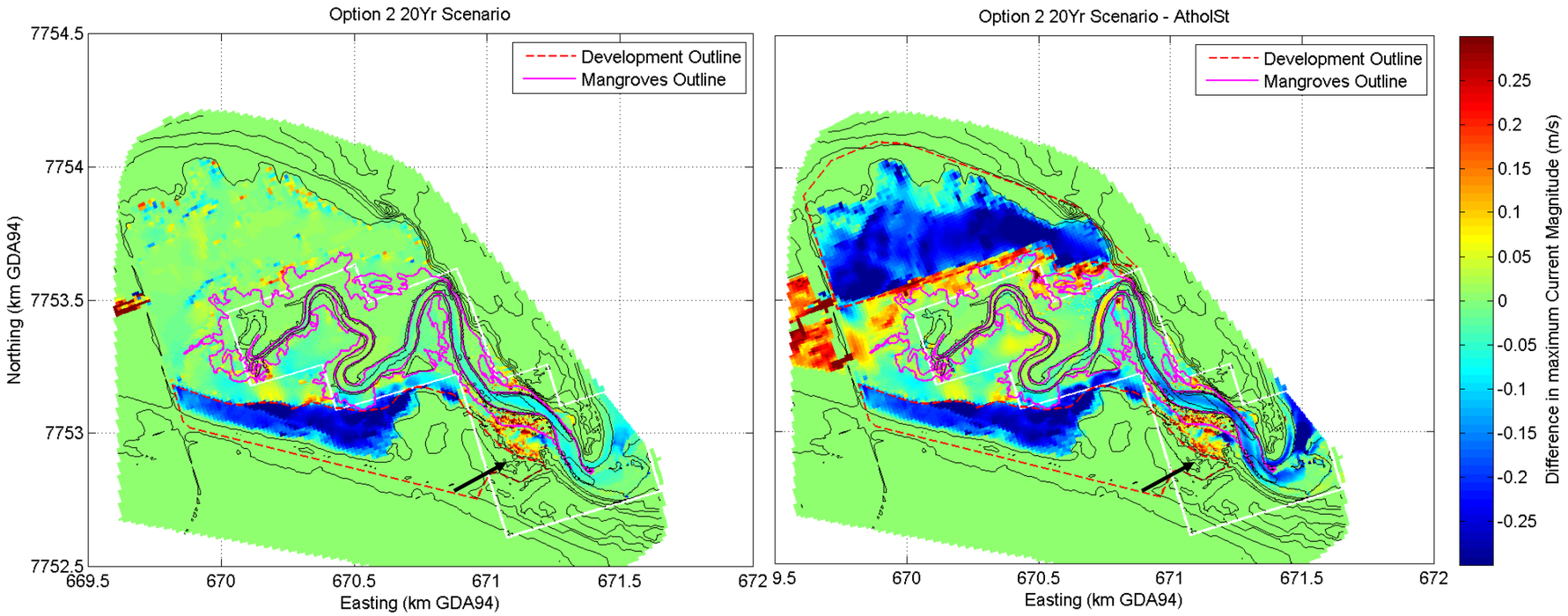

Figure 2-16 Differences in maximum current magnitude for Scenario 3 - Design and Athol Design Cases. Positive change indicates current magnitude is greater for Design Cases. White outline is boundary between domains. Arrow indicates current variability at entrance due to modelled wetting/drying processes

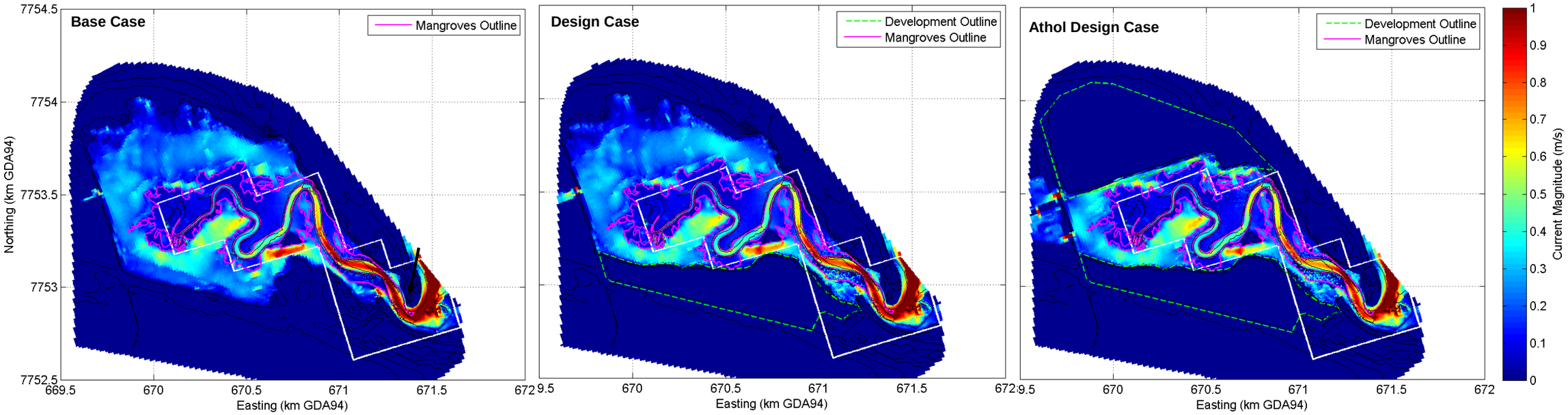

Figure 2-17 Maximum current magnitude for Base, Design and Athol Design Cases for Scenario 3. White outline is boundary between domains

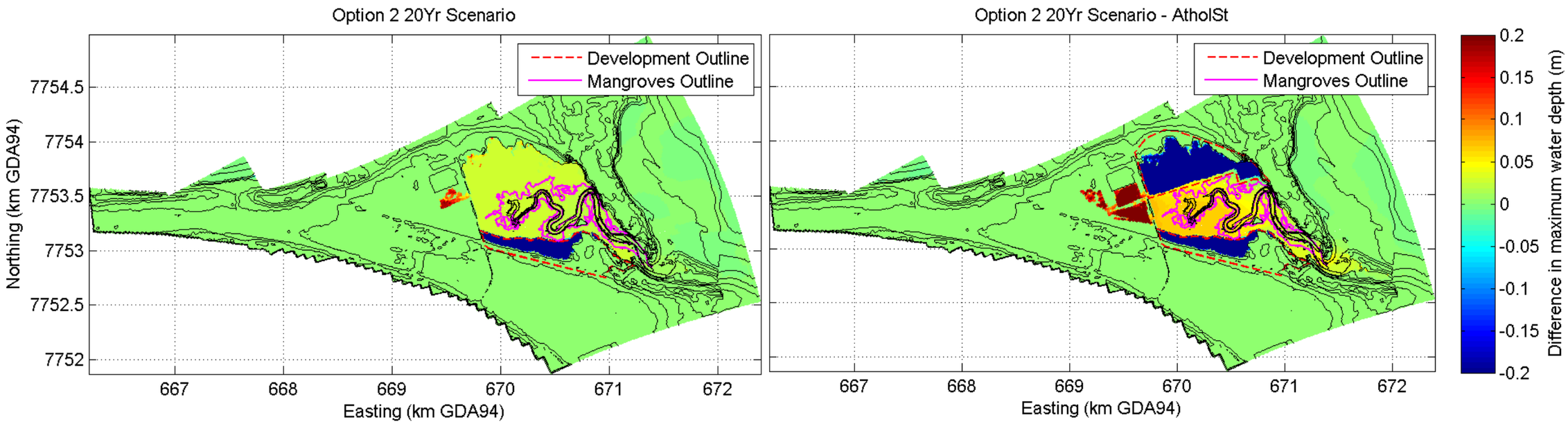

Figure 2-18 Differences in maximum inundation depth for Scenario 3 - Design and Athol Design Cases, zoomed-out view. Positive change indicates depth is greater for the Design Cases

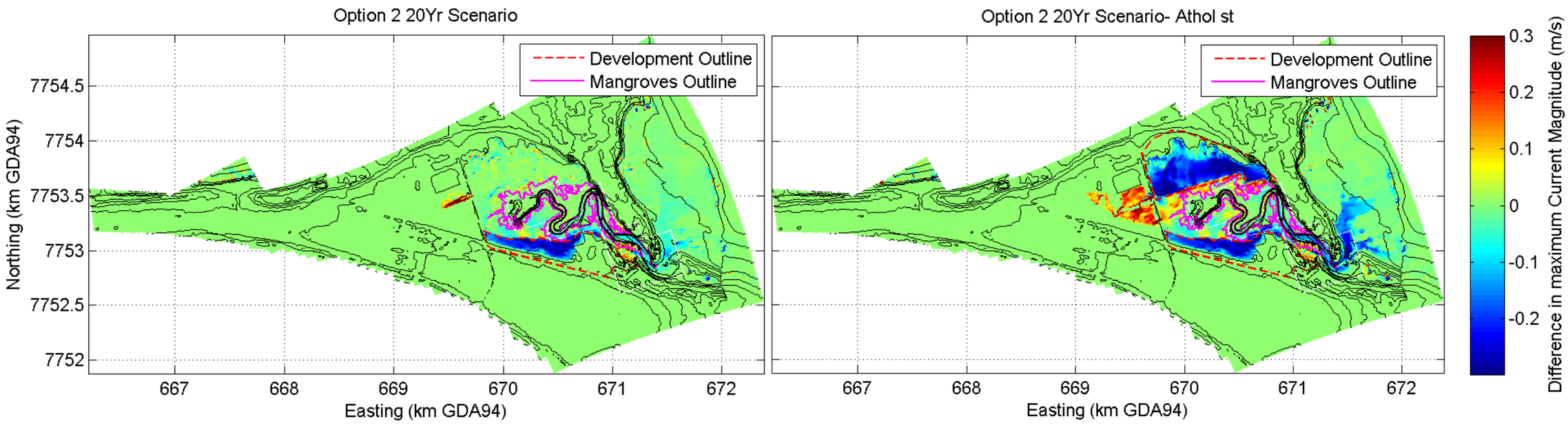

Figure 2-19 Differences in maximum current magnitude for Scenario 3 - Design and Athol Design Cases, zoomed-out view. Positive change indicates current magnitude is greater for the Design Cases

2.5.5 Scenario 4: 500-year ARI Event

The difference between the existing and developed cases in the modelled maximum inundation depth, for the 500-years ARI simulation, is presented in Figure 2-20. Overall, water depth changes in the mangroves are predicted to be minimal, however over an area adjacent to the creek entry, water depths are predicted to be up to 7 cm higher during this event. As discussed in Cardno (2014), this is due to an effective reduction in the 'total' channel width, during extreme events within this region (first demonstrated at the 20-year ARI event). The reduced width on the low friction salt flats (indicated by the arrow in Figure 2-20) increases the flow through the higher friction mangroves, which results in an increased water level required to drive the flow through this area. The selection of Option 2 has limited the increase in water depth to 7 cm for the Design Case, as opposed to the 15 cm observed with the original concept footprint.

The presence of the Athol St development reduces the area available for inundation over the peak of the event. The reduced inundation area results in a reduction in energy dissipated over the tidal flats. This allows the water level within Pretty Pool to reach a higher level over the peak of the inundation event. This effect due to the Athol Street development results in an increase in water depth across Pretty Pool of approximately 5 cm. This combined with the influence of the Stables developments shows a 15 cm increase in water levels near the entrance to Pretty Pool.

The maximum water depth for each of the Base, Design and Athol Design Cases are plotted in Figure 2-21 to enable the depth changes to be placed in context. The changes in water depth in the channel correspond to a change of approximately 3% and 6% for the Design and Athol Design Cases respectively. The change to the depth in the other mangrove areas for the Athol Design Case is also approximately 3%.

The difference in current magnitudes and the maximum current magnitudes over the duration of the storm for the Base, Design and Athol Design Cases are plotted in Figure 2-22 and Figure 2-23 respectively. The maximum current magnitude through the mangroves is approximately 5 cm/s to greater than 100 cm/s for the existing layout.

Maximum current magnitudes in the proximity of the mangroves are predicted to increase by up to approximately 15 cm/s for the Design Case. The Athol Design Case predicts increases of up to 30 cm/s, which will likely result in sediment redistribution during extreme events.

As discussed in Section 2.5.4, the Base Case model results indicate that conditions for sediment suspension are evident in the 20-year ARI Scenario. As expected, conditions for the 500-year ARI scenario also cause sediment suspension. An increase of 15 cm/s and 30 cm/s due to the Design Case and Athol Design Case respectively will result in alteration to the sediment redistribution that occurs during these extreme events. This may result in slight redistribution of the mangrove habitat, however given the frequency of these events it is likely that other direct impacts (i.e. wind) will likely be more significant than the alterations to the hydrodynamic regime due to the development.

For the Design Case, there is negligible change in the overall depth of flooding for areas adjacent to the development (Figure 2-24). This is in contrast to the results of the Athol Design Case (and that assessed in Cardno 2012) where additional flood depth (up to 15 cm) is exhibited adjacent to the rail loop to the west (Figure 2-24). This is due to the overall smaller development footprint of the Design Case within the tidal flats compared to both the Athol Design Case and the Athol Street development assessed in Cardno (2012). For the Athol Design Case, the depth is predicted to increase by up to 15 cm. These changes are shown in Figure 2-24 and Figure 2-25 for the water depths and currents respectively.

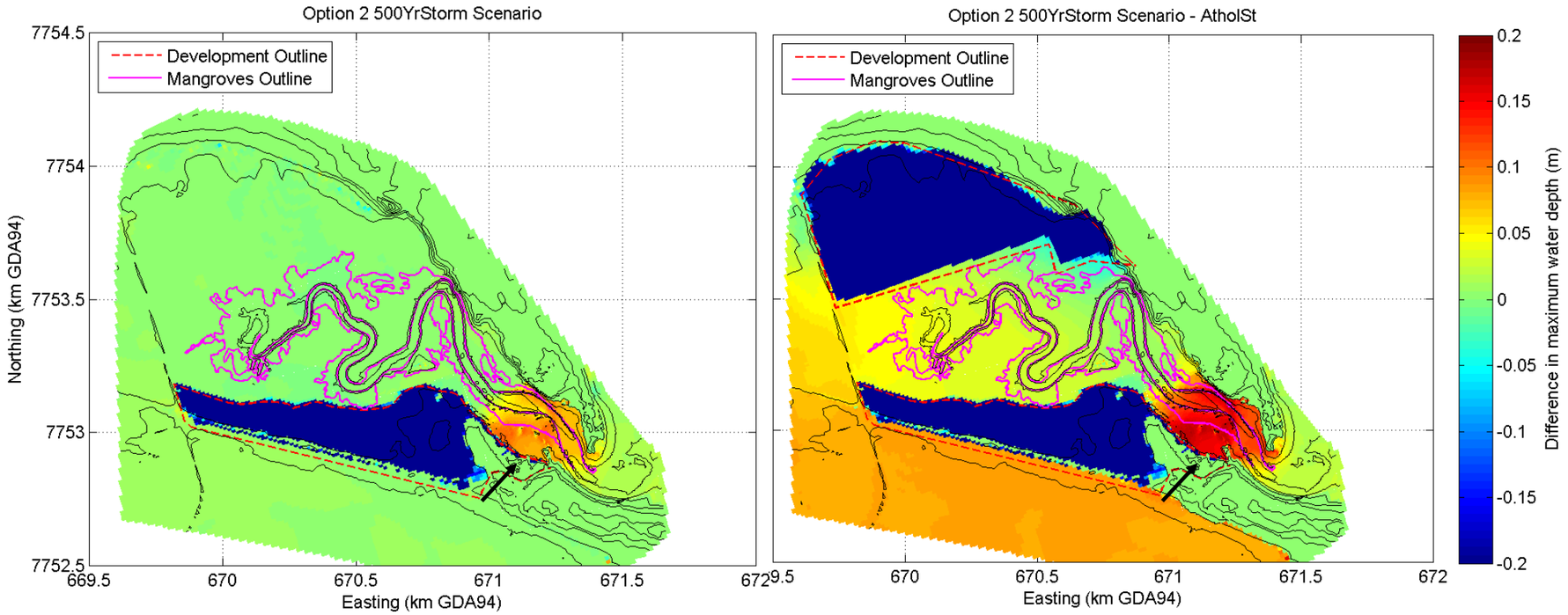

Figure 2-20 Differences in maximum inundation depth for Scenario 4 - Design and Athol Design Cases. Positive change indicates depth is greater for Design Cases

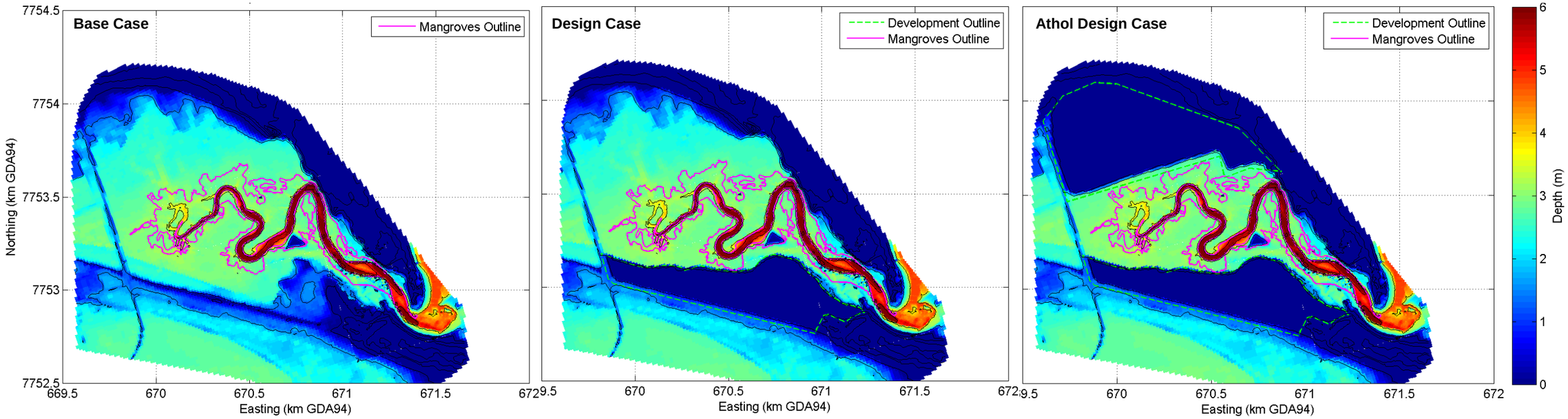

Figure 2-21 Maximum water depth for Base, Design and Athol Design Cases for Scenario 4

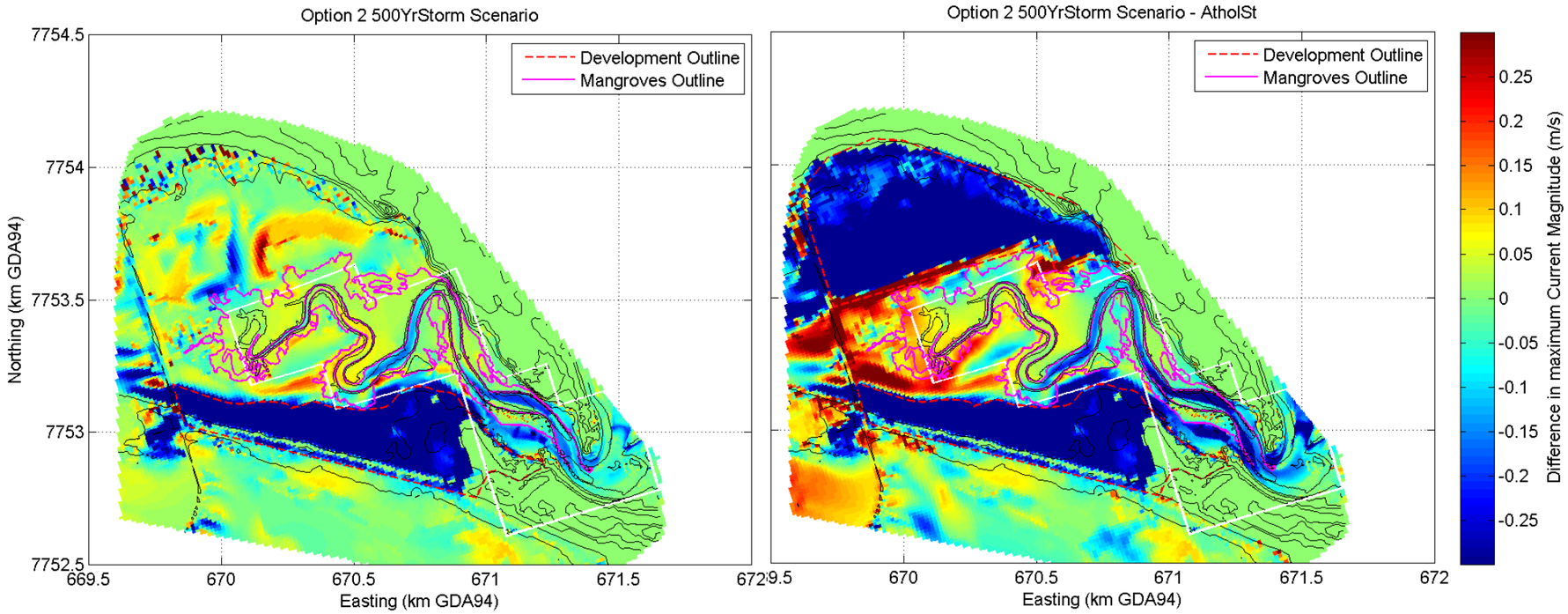

Figure 2-22 Differences in maximum current magnitude for Scenario 4 - Design and Athol Design Cases. Positive change indicates current magnitude is greater for Design Cases. White outline is boundary between domains

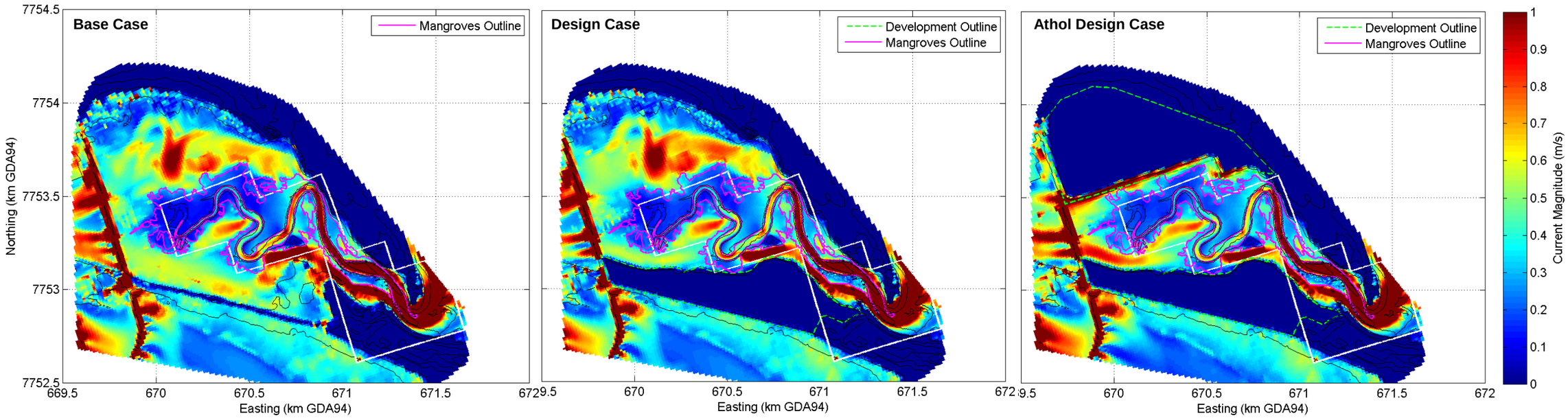

Figure 2-23 Maximum current magnitude for the Base, Design and Athol Design Cases for Scenario 4. White outline is boundary between domains

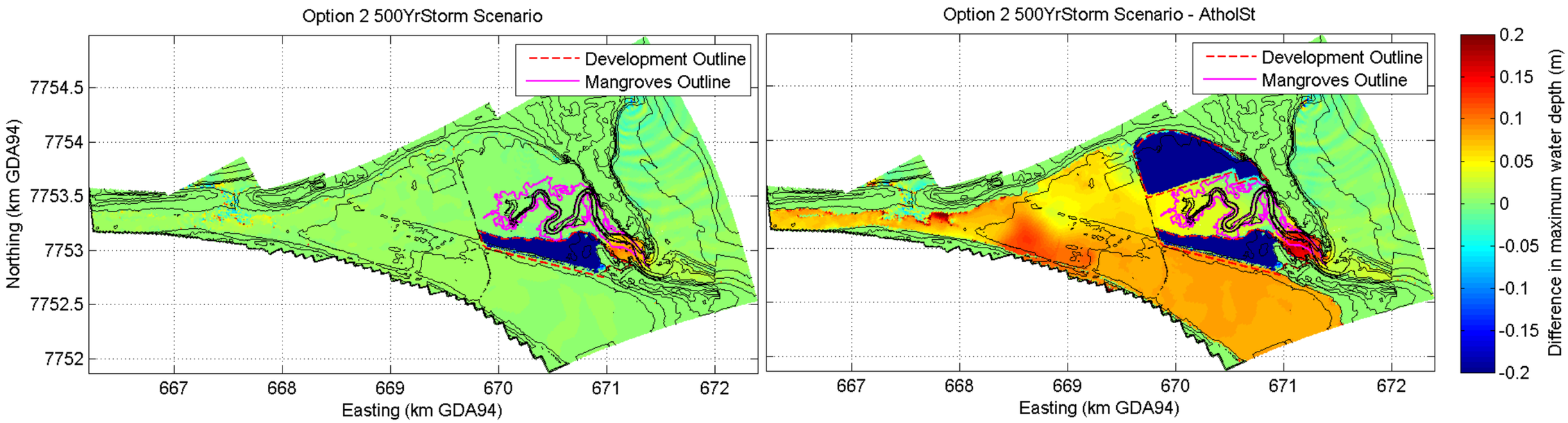

Figure 2-24 Differences in maximum inundation depth for Scenario 4 - Design and Athol Design Cases, zoomed-out view. Positive change indicates depth is greater for the Design Cases

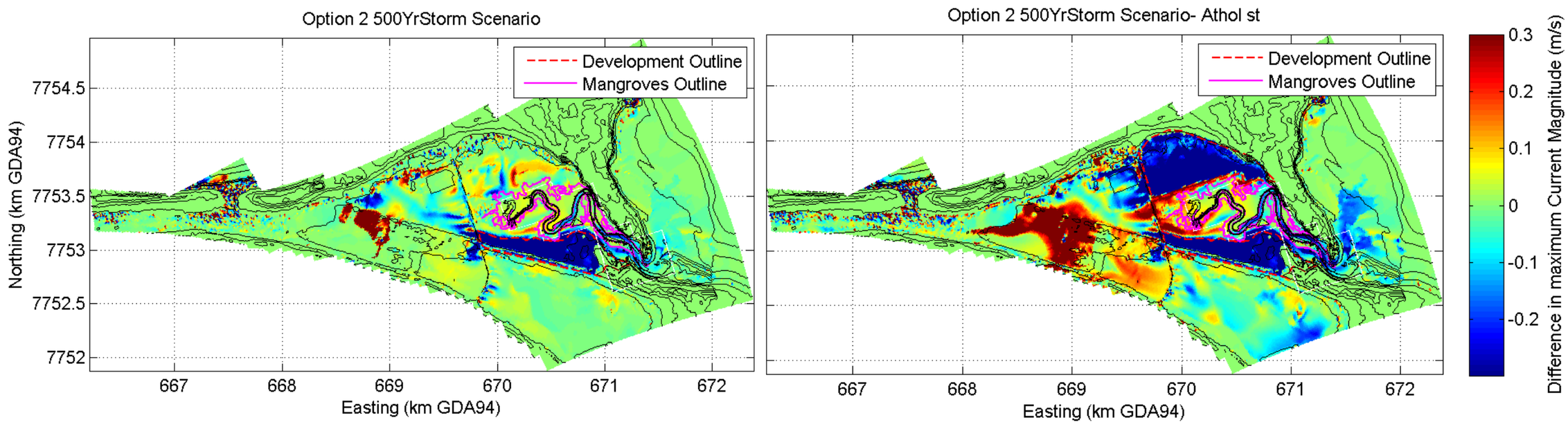

Figure 2-25 Differences in maximum current magnitude for Scenario 4 - Design and Athol Design Cases, zoomed-out view. Positive change indicates current magnitude is greater for the Design Cases

3 Hydrological Assessment

A one dimensional XPSWMM model was developed to assess the existing hydrological environment (Cardno 2014b). The assessment was undertaken to:

> Develop an understanding of how stormwater is currently managed onsite.

> Determine if the site acts as a drainage sump to surrounding land.

> Ensure that if the site is acting as a sump to surrounding land that sufficient space is set aside for stormwater in the masterplan.

The modelling found that currently stormwater flows from the high ridgeline in the south of the site north towards Pretty Pool Creek. There is also a small amount of run-off which flows south towards Styles Road. In larger storm events from the 10 year to 100 year ARI storm event there will be sheet runoff across the site from south to north into the Pretty Pool Creek. Under the existing site conditions no run-off is held onsite.

The site does not act as a drainage sump to surrounding land. Although run-off from the residential land to the south east does flow onto the site, it flows to the north of the site into Pretty Pool Creek.

The volume of run-off predicted to be produced across the site is presented in Table 3-1 for the 10-year and 100-year ARI events.

Table 3-1 Run-off volumes at the development site

Flow Direction	6 minute 5-year ARI (m^3)	10-year ARI (m^3)	100-year ARI (m^3)
North to Pretty Pool Creek	1,200	6,050	11,300
South to Styles Road	250	1,050	1,950

The modelling volumes presented in Table 3-1 give an indication of the amount of water the site generates in each storm event. If insufficient measures are taken during development, these volumes would increase due to the introduction of more impermeable surfaces. It is anticipated however that post development run-off generated during these events will decrease due to the requirement of detaining and infiltrating the first 15 mm of any rainfall event. The anticipated volumes provided in Table 3-1 do not account for the 5 m/day infiltration rate provided by the geotechnical investigation for use in the design of soakwells.

4 Mangrove Impact Assessment

The proposed development has the potential to have both positive and negative impacts on the mangrove community adjacent to the development. The development site includes a portion of supra-tidal salt flats which are flooded about once a month during the spring tides. Whilst the development footprint does not directly intersect with mangrove vegetation, there is still potential for impacts to mangroves due to the proximity of the development from alteration of the tidal prism during high water level events, and through altered groundwater and surface water regimes.

RPS provided Cardno with a spatial map of vegetation types identified through a site visit in March 2010. These are presented in Figure 4-1. The mangroves were observed to comprise of a low closed forest of *Avicennia marina* on the tidal mud flats, with *Rhizophora stylosa* along the limestone embankment and creek line of Pretty Pool Creek. The outline of mangroves in the figure corresponds to the outline shown by the pink line in the model result plots.

There is potential for impacts on the mangroves associated with the decreased tidal prism as a result of changed bathymetry in the development area, due to changes to the current velocity through the mangrove area, and changes in mangrove inundation level and duration. Mangrove distribution in the Pilbara is dependent on a number of factors, including:

> Frequency of inundation (height in the tidal profile, connectivity with the ocean)
> Sediment type, substrate and grain size
> Salinity
> Drainage
> Current speed
> Wave height (Semeniuk and Wurm, 1987)

It is well documented that mangroves typically grow within a narrow topographical band, in general from the contour representing mean water level (MWL) at 0 m AHD, to the contour for mean high water springs (MHWS). Within this range, the frequency of inundation by seawater is sufficient to flush out the groundwater salinity and waste products and provide mangroves with nutrients. However, below MWL, the frequency of inundation is too great and waterlogging occurs. Above MHWS, mangroves cannot survive because the frequency of inundation is too low and groundwater salinity remains too high.

Increases or decreases in water level have the potential to result in waterlogging or inadequate inundation. Changes to current magnitude may result in changes to creek structure or erosion and accretion within the mangroves which could have a negative impact on health and survival.

High salinities in sediments landward of mangrove habitats in the Pilbara are the result of occasional inundation by high spring tides and evaporation (Paling et al. 2003). In these areas salt flats form, which may be devoid of vegetation or contain samphire vegetation which is very salt tolerant. Mangroves at higher elevations adjoining hyper-saline tidal flats are at the extreme margin of their tolerance and are highly susceptible to even minor alterations in naturally occurring environmental conditions. Reduced rainfall or decreased flushing due to sedimentation of peripheral creek lines can result in higher salinity and therefore a decrease in mangrove condition, whereas increased inundation can lead to decreases in salinity which promote an increase in condition and growth.

Mangroves may also grow along the hinterland margin of salt flats. This is more pronounced in mangroves growing in the wet tropics, however it is also occurs in the Pilbara. Figure 4-2 displays examples of this for the study area. Two examples are shown, one of which shows a naturally-occurring example and the second is an example of how this effect can also be created by structures, such as roads, which are constructed over salt flats. In these areas, fresh ground and surface water flows into the mudflats, reducing the salinity of the sediments to below the tolerance limit for mangrove survival (Semeniuk et al. 1978).

Figure 4-1 Mangrove spatial distribution (figure provided by RPS, 2014)

Previous developments in Port Hedland have resulted in incremental loss of mangrove and other intertidal habitats (EPA 2011). This has either been through direct impacts such as clearing, or indirect impacts such as altered hydrology, causing either ponding or reduction in seawater inundation. For example, the construction of causeways that cut off creek lines or impound seawater. Impacts have also been from more subtle long-term effects associated with altered groundwater hydrological regimes, for example impacts resulting from the construction of large scale solar salt ponds (AECOM 2005).

Increased growth of mangroves has also occurred in areas where the salinity of salt flats has been reduced, for example from the leaking of seawater from ponds, or the construction of channels across the salt flats, allowing the inundation of these areas by seawater. This has occurred at the Dampier Salt Eastern Lease Site from the construction of the bitterns channel (Maunsell AECOM 2006).

Figure 4-2 Examples of mangroves growing along the hinterland margin in the Study Area (Image source – Nearmap * Google Earth).

4.2 Impacts Suggested by Results of Hydrodynamic Modelling

Hydrodynamic modelling of the proposed development typical tidal scenarios: 1-month ambient and 2-year ARI spring tide (Scenarios 1 & 2) has shown the following impacts on hydrodynamics for a typical tidal scenario:

> Very minor reduction (<0.5 %) in volume of water flowing into and out of Pretty Pool Creek across a 1-month tidal cycle (Table 2-1).

> Mostly comparable maximum water levels and current magnitudes across the mangroves for pre- and post-development scenarios for the typical tidal scenario.

> Little or no decreases in in maximum inundation water levels within existing mangrove habitat which may cause a negative impact on health and survival. Water levels are predicted to be very slightly higher and

therefore extend further into the salt flats to the north of the study area for the Design Case, however these are outside the vegetated mangrove areas and, given that they are high in the tidal profile, would tend to reduce the salinity of the salt flats and promote mangrove growth rather than cause negative impacts along the landward margin of the existing vegetation. Similarly for the Athol Design Case, maximum water levels are predicted to be slightly higher (3 cm); however this would also tend to promote mangrove growth on the seaward edge of the salt flats.

> Little or no increases or decreases in current magnitude predicted within existing mangrove habitat.

> Small increases in current magnitude (up to 5 cm/s) are predicted in some salt flat areas immediately adjacent to the development area. There is potential for this to encourage the development of drainage and promote the colonisation of mangroves along the edge of the channels.

Impacts on mangroves are therefore considered likely to be minimal as a result of hydrodynamic changes during typical tidal conditions for either the Design Case or the Athol Design Case. It is possible that increases in water levels and current magnitude on the salt flats leads to development of new creeks and therefore a landward expansion of mangrove habitat in this area.

Results of hydrodynamic modelling for a 20-years and 500-yrs ARI Cyclone Scenario, has shown the following impacts on hydrodynamics for an extreme event:

> Up to 5% reduction in volume of water flowing into and out of Pretty Pool Creek for the Design Case, and up to 16 % reduction for the Athol Design Case.

> Small increases in maximum inundation water depths of up to 5 cm across the mangroves and salt flats during the 20-years ARI storm conditions for the Design Case and 7 cm for the Athol Design Case, which are likely to be inconsequential given the transient nature of the conditions.

> For the Design Case, maximum inundation water depths were unchanged for the majority of the study area from the Base Case during a 500-years ARI Cyclone Scenario. However, there was an area of mangroves towards the mouth of the creek that is predicted to experience increased water levels of up to 7 cm.

> For the Athol Design Case, the majority of the study area showed an increase of up to 5 cm maximum water levels during a 500-years ARI Cyclone Scenario, and the mangroves towards the mouth of the creek are predicted to experience increased water levels of up to 15 cm.

> The 500-years ARI Cyclone Scenario is predicted to have a considerable effect on the current flows in the study area for the Base Case, the Design Case and the Athol Design Case. High current speeds are predicted for the salt flats between the mouth of Pretty Pool and the Stables development footprint (Figure 2-23), which would likely result in mobilisation of sediments of the salt flat. Currents through within the mangroves are considerably less, and mostly similar for the Base Case, the Design Case and the Athol Design Case in this area.

> The Design Case and the Athol Design Case results in increased current speeds of 15 cm/s and 30 cm/s respectively towards the western end of the mangroves and salt flats (Figure 2-22). This results in increases in current speed of up to 15 cm/s and 30 cm/s for the Design Case and the Athol Design Case respectively. While this is a relatively large increase, current speeds in the mangroves are low in comparison to those predicted for the salt flats (Figure 2-23). The Base, Design and Athol Design Cases are all likely to result in mobilisation of salt flat sediments leading to damage to the mangroves in this area for this scenario, with slightly increased currents due to the development predicted.

On the basis of these results, an increase in the severity of impacts on mangroves as a result of hydrodynamic changes during extreme events are possible; however, these are likely to be minimal and insignificant in comparison to likely wind damage at such times.

4.3 Impacts Suggested by Results of Hydrological Assessment

The potential for impacts to mangroves from altered groundwater and surface water regimes was investigated using the results of the Local Water Management Strategy (Cardno 2014b). The results from this suggest that there will be decreased surface runoff from the developed site in comparison to the existing salt flats and the hinterland. None the less, storm water discharges are predicted (Figure 4-3) to flow into the mangroves. The predicted decrease is a result of the increased permeability of the fill material than the existing soils. However, given the relative permeability of underlying layers, groundwater discharge is likely to occur at the base of the fill area where it meets the salt flats.

This could in fact result in an overall increase in freshwater onto the tidal flat. It is likely to create a hinterland effect at the base of the development bund wall and potentially allow for colonisation of mangroves along the wall and on the margins of any drainage channels forming due to the altered patterns of freshwater run-off.

The water budget of the site is also likely to be changed due to altered land use. Residential and public open space will be irrigated using scheme water from offsite, potentially leading to increased infiltration to groundwater. Nutrients discharged from these areas have the potential to lead to increases in growth of mangroves.

The additional freshwater and nutrients associated with suburban land use also has the potential to lead to increased cyanobacterial mat (algal) growth on the salt flats (Paling et al. 1989) which may be visually apparent from the development area.

There is a possibility that the altered surface flows and the weight of the development could create a hydrostatic head and alter groundwater flows such that hyper-saline groundwater associated with salt flats under and adjacent to the development moves towards the mangroves. When groundwater salinity exceeds mangrove tolerance limits, negative impacts on mangrove health can occur along the salt flat / mangrove vegetation boundary. This phenomenon has been observed in the region in the past in relation to the placement of solar salt ponds on supratidal mudflats (AECOM 2005). However, there is no evidence of this occurring from similar historical land developments in the area, and increases in freshwater flow from the development are likely to counteract this potential impact. This is therefore considered to be an unlikely impact.

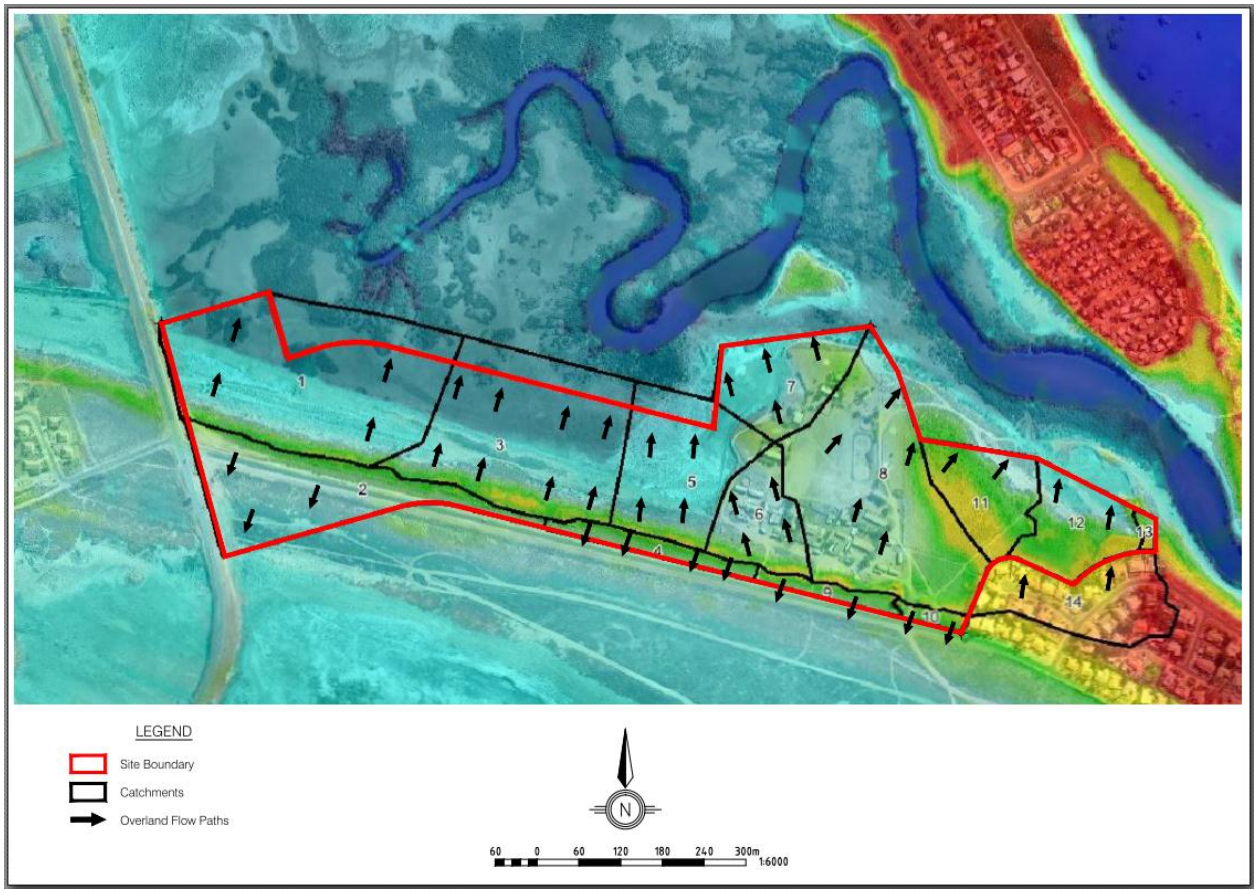

Figure 4-3 Surface water flow paths (Cardno 2014b).

4.4 Conclusions

The potential impacts associated with the development are summarised in the matrix below, Table 4-1.

Table 4-1 Mangroves impact and effect matrix

	Mechanism for positive impacts	Mechanism for negative impacts	Conclusions
Water Level	Increased water levels associated with storm events may increase flushing of hyper-saline flats and increase area suitable for mangrove growth.	Increased water levels during extreme storm events	Water level differences during typical conditions are patchy and of small magnitude. Therefore water level changes are considered likely to have minimal impact on mangroves. An increase in the severity of impacts on mangroves as a result of hydrodynamic changes during extreme events is possible; however, these are likely to be minimal and insignificant in comparison to likely wind damage at such times.
Current Flows	Increased flows during storm events may lead to erosion of salt flat sediments, creation of new drainage lines, reduction in salinity and increase in area available for mangrove recruitment.	Negative impacts to mangroves near the mouth of the creek are indicated during extreme storm events (cyclones).	Direct cyclonic impacts on vegetation likely to be greater than effects from altered hydrodynamics associated with the development, therefore impacts on mangroves associated with altered current flows are also considered to be minimal. As for water level increases, an increase in the severity of impacts on mangroves as a result of hydrodynamic changes during extreme events is considered to be insignificant in comparison to likely wind damage at such times.
Groundwater Salinity	Increased localised freshwater flows due to hinterland effect, stormwater drainage and altered land use. Localised freshwater input is predicted to result in a decrease in groundwater salinity (and increased nutrient concentrations) in tidal flats, potentially promoting mangrove colonisation and growth, particularly on the salt flat along the development margin.	Altered hydrology and weight of development may cause hydrostatic head and alter groundwater flows such that hyper-saline groundwater under and adjacent to the development moves towards mangroves.	Mangrove recruitment along the bund wall is predicted. Mangrove condition on the seaward margin of the salt flats may improve due to decreased salinity associated slight increases in inundation and current flow, conversely there is potential for delayed negative impacts on creek mangroves along salt flat margin. On balance, it is considered most likely that the development will decrease the salinity of the salt flats and promote the survival and growth of mangroves.
Nutrients	Nutrients introduced by altered land use may result in increased growth in mangroves.	Nutrients may cause increased cyanobacterial mat (algal) growth on the salt flats which may be visually apparent from the development area	Both positive and negative impacts from increased nutrients are likely to be minimal.

5 Erosion Potential Investigation

5.1 Geomorphic Setting

The general coastal morphology of the Port Hedland region is a limestone barrier system which is typified by the low coastal cliff and rock outcrop formations along the shoreline. In general, there are limited mobile sediments present at the shoreline, however, the development of the Spoil Bank from dredge spoil since the 1960's has provided a source of mobile sediment. Eliot et al (2013) found the Port Hedland area to consist of one secondary compartment with four tertiary sediment cells; the study area lies in the Beebingarra cell. A sediment cell is considered to be an area where sediment movement outside the cell is largely constrained. The area was found to differ from the 'typical' Pilbara behaviour described by Semeniuk (1996), as there is limited fluvial sediment input, although there is some sediment supply through coastal transport.

The coastline is identified as a mixture of Pleistocene and more recent overlying Holocene formations (Semeniuk, 1996). The large tidal range combined with wave forcing provides a highly energetic environment near the shoreline, which results in limited mobile sediments remaining in the nearshore zone. Sandy beaches in the Port Hedland region are normally perched on rock platforms or are constrained by rock formations. As a result of the underlying rock along much of the coastline, the mobility and erosive potential is limited under the combined effects of waves and currents.

Cooke Point is located at the narrowest point in the tidal flats along the Beebingarra cell, in an area of limited sediment. The sediment in the inter-tidal terrace is highly dynamic. Changes to sediment availability have a significant impact on these types of areas. The analysis conducted by Eliot et al (2013) noted a reduction in sediment availability in the last 50-years, and the onshore migration of the bar immediately offshore from the study area. This could be due to changes up-drift, or due to modifications to the tidal creek networks, including roads and drainage pathways.

Adjacent to Pretty Pool Creek the beach is backed by a vegetated dune system and fronted by sand flats extending up to 700 m offshore.

Mobile sediments in the nearshore zone generally have an eastward movement (translating to southerly in the study area) although this can vary seasonally. The predominantly eastward sediment transport in the nearshore zone at Port Hedland is illustrated by the movement of the Spoil Bank since its formation (Figure 5-1). Eliot et al (2013) suggested the recent spit formation in the vicinity of Pretty Pool could be contributed to sediment supply from the Spoil Bank formation.

The Spoil Bank extends out from the coastline as a sand-spit, parallel to the Newman/Goldsworthy channel. The spoil bank is a major man-made nearshore feature that has developed as a result of dredge spoil from the development of Port Headland Inner Harbour and navigation channel.

Dredging commenced from the mid 1960's when the first iron ore berths were developed. Expansion of the inner harbour facilities and subsequent deepening to accommodate Cape Class vessels required major capital dredging programs in the years 1976 to 1977 (1.9 million m^3) and 1984 to 1987 (12.7 million m^3). Ongoing maintenance dredging has occurred at three to four-year intervals with 600,000 m^3 removed in each of the periods 1998 to 1999 and 2000 to 2001 (Paul, 2001).

The northern end of the spoil bank curves to the east suggesting that eastward sediment transport is the dominant sediment transport direction, associated with the prevailing swell direction. The evolution of the spoil bank between the years 1949 to 2009 is shown in Figure 5-1.

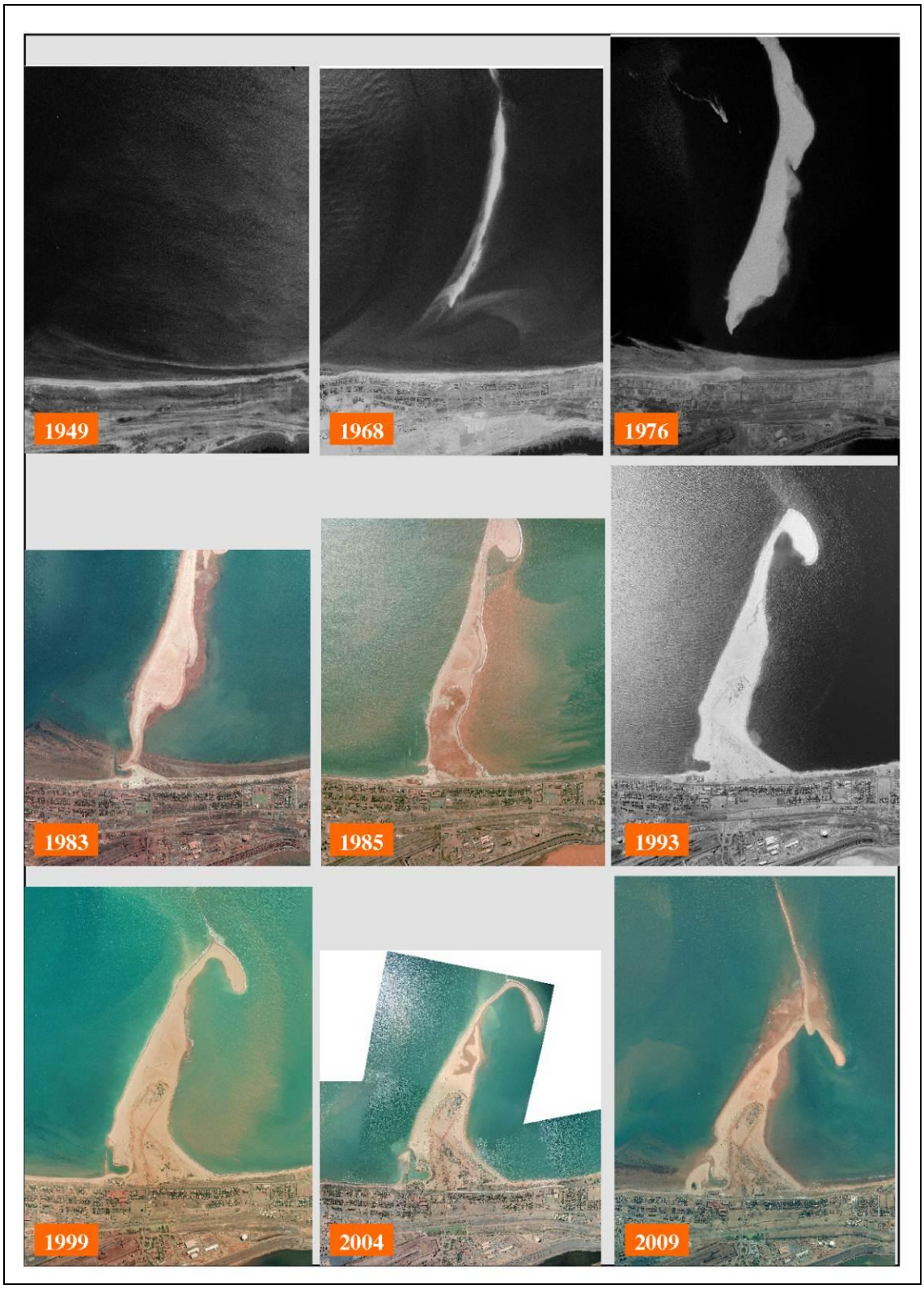

Figure 5-1 Evolution of the Spoil Bank 1968 to 2009 - Port Hedland (Cardno 2011)

5.2 State Coastal Planning Policy (SPP2.6)

The WAPC gazetted the latest SPP2.6 in July 2013. The purpose of SPP 2.6 is to provide guidance for decision making within the coastal zone, including managing development and land use change. Schedule One of SPP 2.6 provides guidance for calculating the coastal foreshore reserve to allow for coastal processes including present day erosion, historical shoreline movement, sea-level rise and storm surge inundation. The component of the coastal foreshore reserve to allow for coastal processes should be sufficient to mitigate the risks of coastal hazards by allowing for landform stability, natural variability and climate change. The coastal foreshore reserve is a critical input into the coastal hazard risk management and adaptation planning framework outlined in SPP 2.6.

This study examines the effects of predicted coastal erosion on the stability of the creek entrance. The area to be assessed is treated as sandy coast, as per the policy recommendations. The allowance for erosion on sandy coasts is calculated as the sum of the S1, S2 and S3 Erosion components, plus 0.2 m per year allowance for uncertainty, and should be measured from the horizontal shoreline datum (HSD):

1. (S1 Erosion) Allowance for the current risk of storm erosion
2. (S2 Erosion) Allowance for historic shoreline movement trends
3. (S3 Erosion) Allowance for erosion caused by future sea level rise

SPP 2.6 states that the coastal foreshore reserve should be defined on a case by case basis including S1, S2 and S3, where relevant. Each of these components is assessed in this report by the following methods:

- S1 Erosion: Selected profile modelling of storm erosion using the storm erosion model system SBEACH (Storm-Induced Beach CHange).
- S2 Erosion: Analysis of historical aerial photography to establish historical shoreline changes.
- S3 Erosion: Application of an allowance of 90 m of shoreline recession based on a vertical SLR of 0.90 m over the 100 years planning horizon to 2110.

The coastal foreshore reserve is applied from a horizontal shoreline datum (HSD), a fixed line that is defined on the basis of the type of coastline being assessed. The HSD defines the active limit of the shoreline under storm activity, and should be determined against the physical and biological features of the coast. In most cases it should be defined as the seaward shoreline contour representing the peak steady water level under storm activity.

Schedule One of SPP2.6 describes different areas for the definition of the storm event for use as the defined storm event in the assessment of inundation and erosion. The Port Hedland region lies in an area of the Western Australian coast that is affected by severe tropical cyclones. Policy guidance for coastal erosion is that a cyclone event corresponding to the 100-years ARI ocean forces and coastal processes should be selected, tracking to maximise its erosion and inundation potential.

Included in SPP 2.6 is the current policy relating to the Sea Level Rise (SLR) projection for the 100-years planning period up to 2110. This has been adopted for this study and +0.9m for a 100-years (2110) planning period.

5.3 Allowance for the Current Risk of Storm Erosion (S1 Erosion)

Storm-induced erosion was investigated in the study area using the SBEACH numerical model as part of the PHCVS (Cardno, 2011). Results were extracted for Transects 6 and 7 from the PHCVS, as shown in Figure 5-2. SBEACH (Storm-induced BEAch Change) was developed to calculate beach and dune erosion under storm wave action, as described in Wise et al (1995).

The following parameters were input into the SBEACH model:

- Beach profile from the top of the foredune to offshore based on the digital terrain model (approx. +8m to -8m AHD).
- Depth of available sand below the beach profile.
- Depth of underlying rock strata (SBEACH 'hard bottom' feature).
- Sediment grain size.
- Time-series of water-level for a Category 5 Cyclone Event (tide + storm surge).
- Time-series of significant wave Height (H_s) for a Category 5 Cyclone Event.
- Time-series of peak wave period (T_p) for a Category 5 Cyclone Event.

The response of the beach profile was assessed for short-term erosion based on design storms representative of a Category 5 Cyclone. This was selected as per the earlier revision of the State Coastal Planning Policy, current at the time of writing the PHCVS. The 500-year design storm was run three times consecutively. The latest SPP 2.6 recommends the 100-year ARI design storm, as discussed in Section 5.2. For comparison purposes, SBEACH modelling of Cyclone Connie and Cyclone John was additionally undertaken on each SBEACH profile and presented along with the 500-years design storm results. Cyclones Connie and John are two of the three most severe events recorded at Port Hedland as of 2011.

Figure 5-2 SBEACH transect locations from PHCVS

The PHCVS model was run using a 0.3 mm grain size. Subsequent studies by Cardno (Cardno 2013) show this is a conservative simulation as sediment samples taken from the area had a D_{50} of 0.48 to 0.78 mm. The smaller grain size is more mobile so results will potentially show greater erosion than is likely to occur. Sensitivity analysis on grain size conducted in the PHCVS showed that an increase in modelled grain size to 0.4 mm reduced the erosion by 43%.

The change to the shoreline profile from the SBEACH model results were assessed at the vegetation / Horizontal Shoreline Datum (HSD) line (4.2m AHD). The results from the SBEACH model are presented in Figure 5-3 and Figure 5-4. Profile 6 is considered to be more representative of the entrance conditions. The results for the three storms are summarised in Table 5-1 below.

The maximum distance of erosion from the HSD line is predicted to be 19 m for Profile 6 for the 500-year design storm event. For Cyclones Connie and John, this recession is predicted to only be 1 and 2 m respectively. The HAT contour is predicted to move seaward by 4 m for the 500-year design storm, as the dune face slumps. This contour is predicted to recede for Cyclones Connie and John. Note SBEACH assumes the material to be eroded consists solely of sand, and doesn't take into consideration any vegetation or matting that may reduce erosion. The results should be interpreted as a worst case scenario of the potential storm-induced erosion at the site.

The location of the underlying rock structure was set as part of Cardno (2011). The results show the profile eroded back to this location for the 500-year design storm. If this rock structure was not at the location assumed in the model, there could potentially be more erosion experienced at the site. It should be noted that review of aerial photography and landforms strongly suggest that the northern bank of the Pretty Pool Creek entrance is underlain by rock due to the generally stable configuration of this area – see Section 5.4.

Table 5-1 Results From SBEACH Modelling, Profile 6, D_{50} 0.3 mm

Storm Event	HSD Line (4.2m AHD)	HAT Line (3.6 m AHD)
500-year Design Storm	-19 m	+4 m
Cyclone Connie	-1 m	-3 m
Cyclone John	-2 m	-4 m

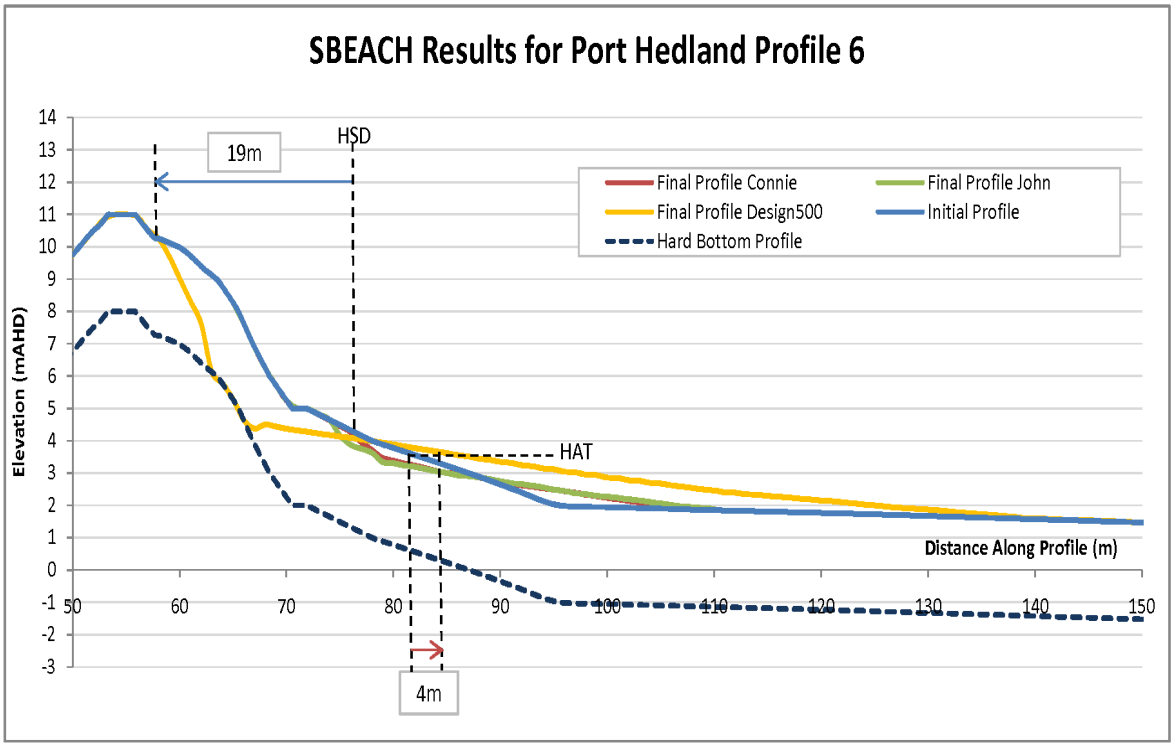

Figure 5-3 SBEACH results for Profile 6, north of the creek entrance

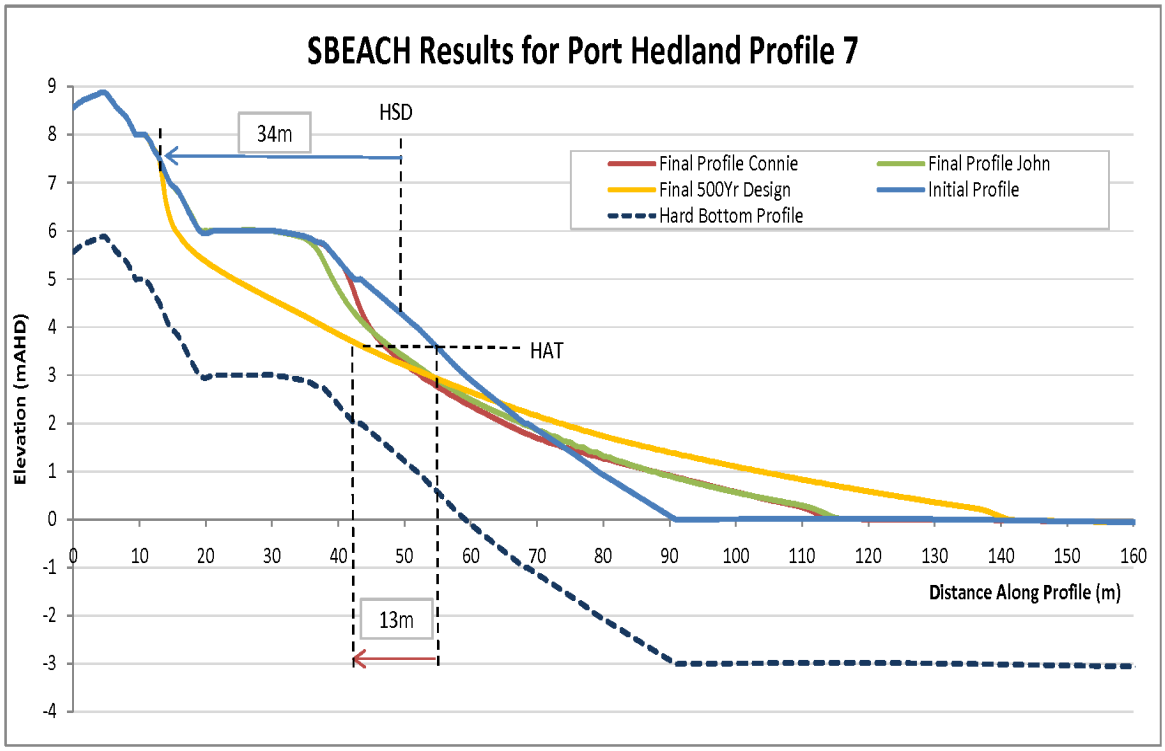

Figure 5-4 SBEACH results for Profile 6, north of the creek entrance

5.4 Allowance for Historic Shoreline Movement Trends (S2 Erosion)

Nine aerial data sets were obtained from Landgate Imagery as part of Cardno (2011) covering the 60-years period between 1949 and 2009 as shown on Table 5-2**.** Major cyclones preceding each of the aerial datasets are also shown.

Table 5-2 Port Hedland Photogrammetric Sources

Date	Source	Resolution	Preceding Major Cyclone Event
1 August 1949	LANDGATE Imagery	1:50000	Unknown
2 June 1968	LANDGATE Imagery	1:80000	Shirley (Mar 1966)
15 November 1976	LANDGATE Imagery	1:34500	Joan (Dec 1975)
1 September 1983	LANDGATE Imagery	1:12000	Jane (Jan 1983)
7 July 1985	LANDGATE Imagery	1:25000	Chloe (Feb 1984)
4 August 1993	LANDGATE Imagery	1:50000	Connie (Jan 1987)
25 July 1999	LANDGATE Imagery	1:25000	Gwenda (April 1999)
1 September 2004	LANDGATE Imagery	1:7500	Monty (Feb 2004)
18 May 2009	LANDGATE Imagery	1:25000	George (Feb 2007)

As part of the PHCVS, photogrammetric analysis was carried out on the aerial photographs. Aerial images were converted to 3D vector data by triangulating survey points within each of the aerial photography datasets. Following this the MHWS and HAT lines were determined within each of the datasets and used to assess shoreline movement.

- Mean High Water Springs MHWS = 2.8m AHD
- Highest Astronomical Tide HAT = 3.6m AHD

Transects were cast at 100 m intervals along the shoreline. Figure 5-5 shows the position of transects along the present study area. For each of the years of photogrammetric data, the transect lines were measured to the intersection point of the MHWS contour. This contour was plotted over the aerial photographs, and the movement over time assessed. For this study, the MHWS contour was extracted from the LiDAR data provided by LandCorp and added to the figure. This assessment is described below in Section 5.4.2.

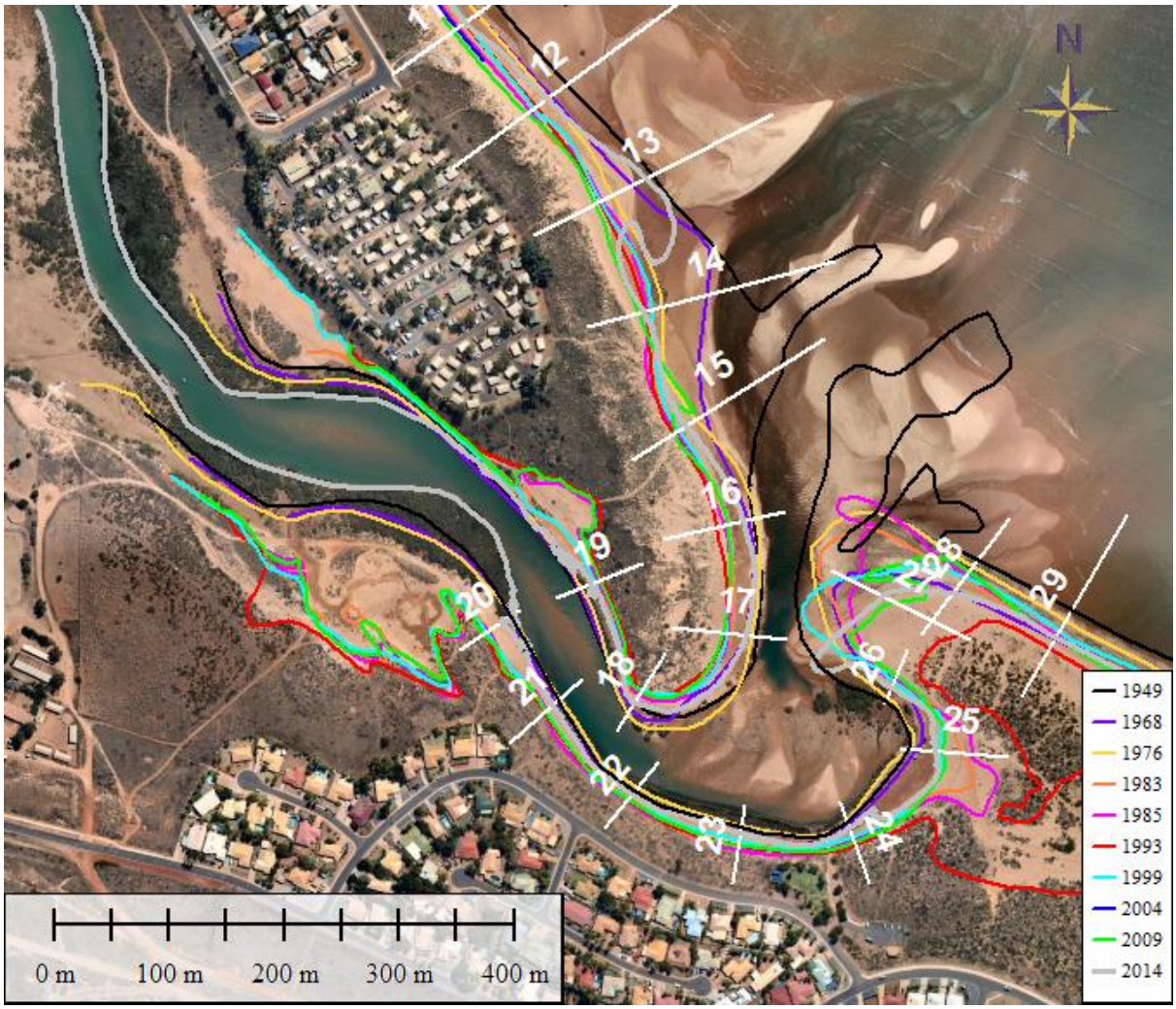

Figure 5-5 Transects 12 to 29, cast at 100 m intervals. Coloured lines indicate the MHWS contour for each aerial photograph year

Shoreline position change was calculated as the difference in transect length relative to the 1985 dataset. The baseline year of 1985 was selected as this was the year in which the spoil bank was established as a connected landform on the Port Hedland shoreline (refer Figure 5-1). The shoreline position by transect is analysed in Section 5.4.3.

5.4.2 Spatial Analysis

Transects relevant for this study area are transects 16 and 17. Figure 5-6 presents these in a zoomed-in view between the year 1949 and 2014. The shoreline position receded in the survey periods 1968, 1976 and 1983. There was very little movement in subsequent years until 2009. The 2014 MHWS contour indicates significant accretion, almost to the 1968 level.

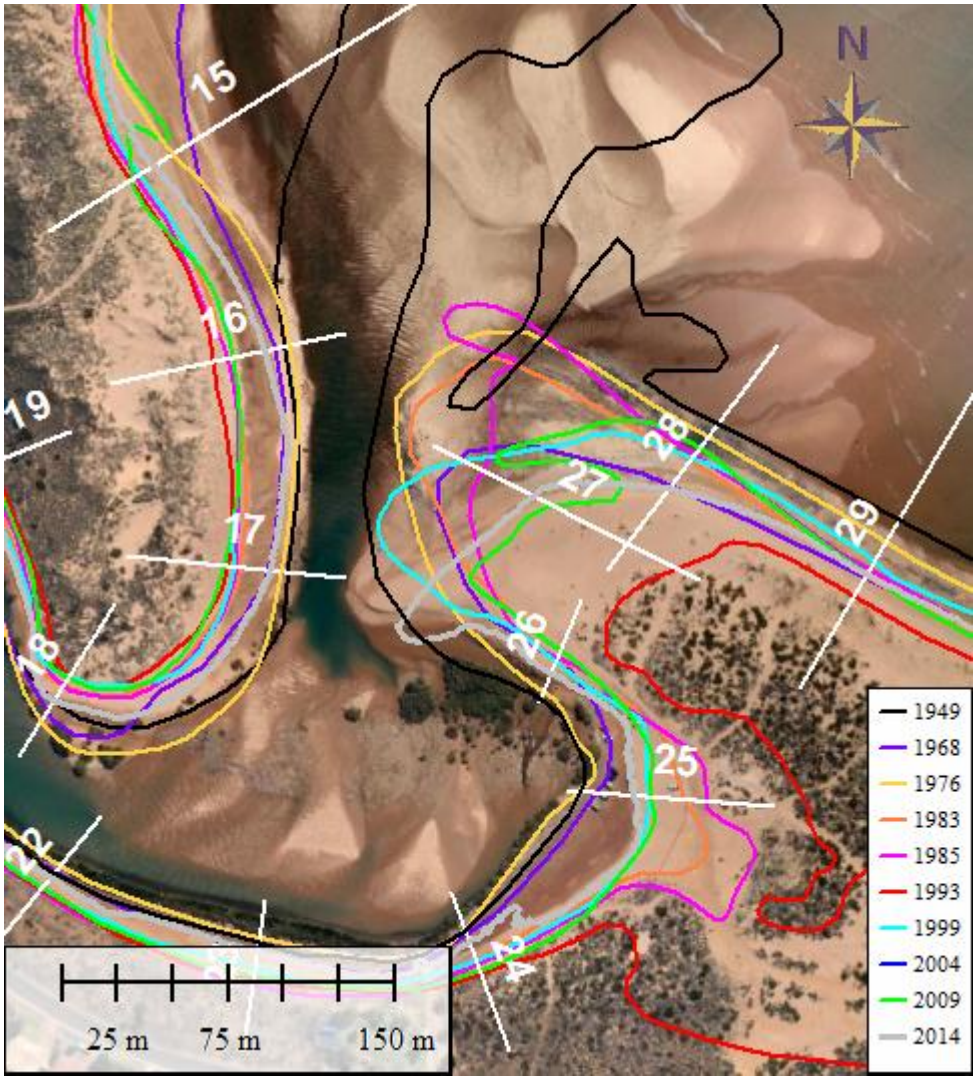

Figure 5-6 Historical shoreline position change for transects 16 and 17

5.4.3 Transect Analysis

The net change in the shoreline position was assessed over the 65-years period from 1949 to 2014 based on the shoreline position in the first and last photogrammetric data sets. To measure change in shoreline position across the study site, each of the transect lines was measured to the intersection point of the MHWS contour for a given photogrammetric set.

The net shoreline change over the period is shown for transects 16, 17 and 28 in Figure 5-7. Transect 28 is included to allow a comparison with the southern side of the creek mouth. All transects show a net recession of the shoreline. Note this movement could oscillate over the time period; this figure indicates the underlying trend. Transects 16 and 17 have a net recession of less than 10 m.

To further understand the timing and causes for the shoreline position changes, a breakdown of the shoreline position change into the years 1949 to 1983 and 1985 to 2014 is shown in Figure 5-8 and Figure 5-9. From 1949 to 1983 the shoreline receded for all transects. From 1985 to 2014, transects 16 and 17, accreted on average by 23 m.

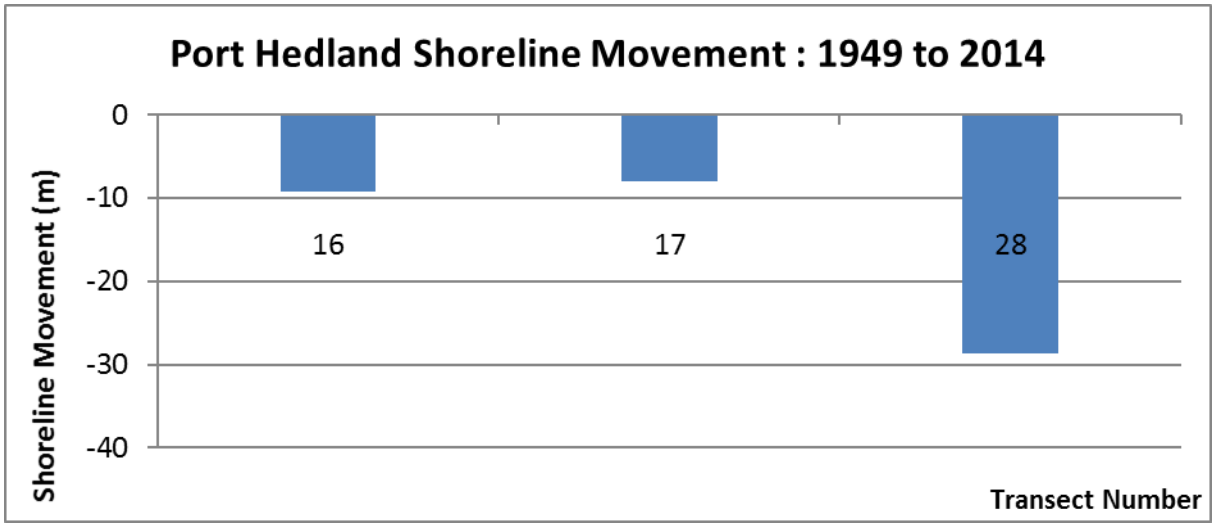

Figure 5-7 Net Shoreline Changes 1949 to 2014

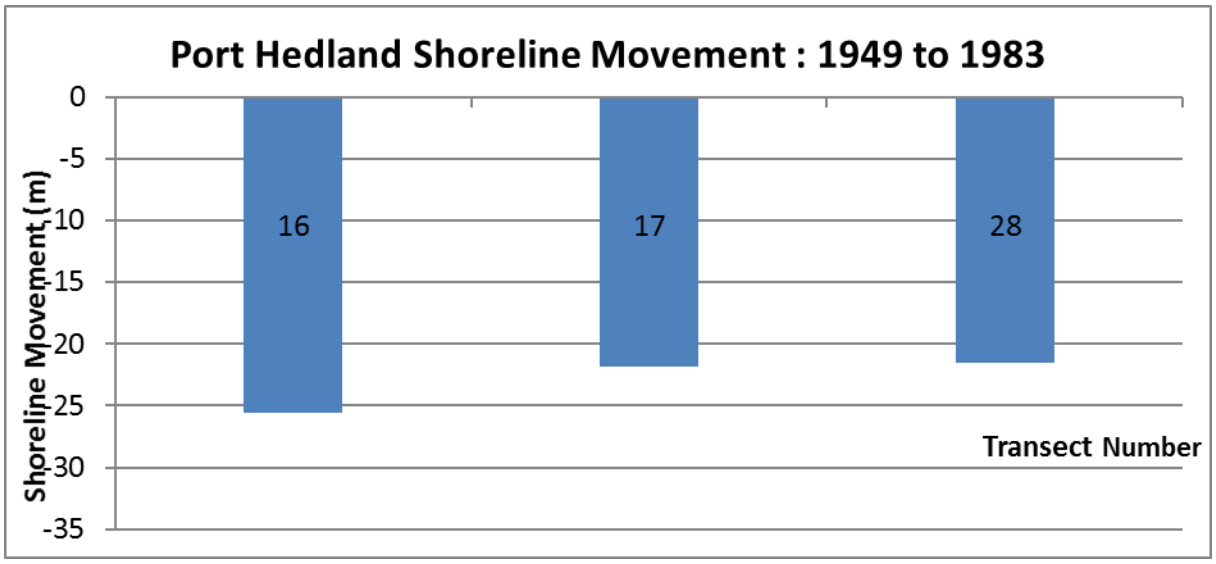

Figure 5-8 Net Shoreline Changes 1949 to 1983

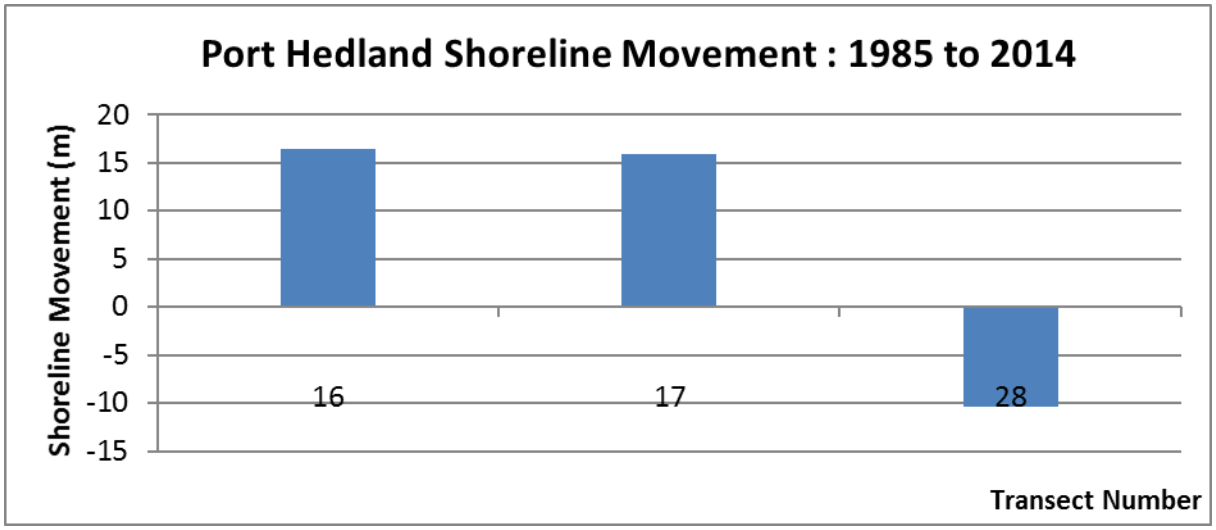

Figure 5-9 Net Shoreline Changes 1985 to 2014

5.5 Allowance for Erosion Caused by Future Sea Level Rise (S3 Erosion)

SPP2.6 recommends a response to sea-level rise of 100 times the vertical sea-level rise. This corresponds to a recession of 90 m for the 2110 planning period.

5.6 Total Erosion Potential

As discussed in Section 5.2, the allowance for erosion on sandy coasts is calculated as the sum of the S1, S2 and S3 erosion components, plus 0.2 m per year allowance for uncertainty. This is presented in Table 5-3.

The S1 value calculated in Section 5.3 is not directly applicable at the mouth of Pretty Pool Creek. The presence of the sand bar system at the mouth will significantly attenuate the wave action resulting in less erosion occurring at the spit than predicted at Profile 6. For an indication of the potential variation, results are presented for Cyclones Connie, John as well as the 500-year design storm.

The S2 allowance presented in Table 5-3 is calculated as 100 times the historic annual rate of erosion, as per SPP2.6 requirements. The maximum historic erosion presented in Section 5.4 at the study site is 10 m over the 65-year analysis period. This corresponds to an S2 value of 15 m.

The S3 allowance is conservative as it assumes the Bruun rule is applicable at the site. It is anticipated that the presence of the bed rock would limit the erosion to a much smaller value.

The average width of the vegetated dune on the spit is 115 m, and the distance from the HSD line to the concept development footprint is 230 m. Whilst applying these methods is not directly applicable at the mouth of the creek, these conservative results provide sufficient confidence that the spit will remain to protect the development site for the 100-year planning period.

Table 5-3 Predicted erosion allowance

S1 Result	Acute Erosion (S1)	Long-term Erosion (S2)	Sea Level Rise Erosion (S3)	Uncertainty Allowance	Erosion Setback (m behind HSD)
500-year Design Storm	19 m	15 m	90 m	20 m	**144 m**
Cyclone Connie	1 m	15 m	90 m	20 m	**126 m**
Cyclone John	2 m	15 m	90 m	20 m	**127 m**

6 Conclusions & Recommendations

A calibrated hydrodynamic model has been developed for Pretty Pool Creek, surrounding mangroves and adjacent township. Simulations suggest that the post-development layout will result in little alteration to the hydrodynamic regime of the mangroves and the creek under normal tidal conditions. During extreme events, the post-development layout results in alterations to the hydrodynamic regime of the mangroves and creek with an average recurrence interval less than 1 in 20 years.

Following initial modelling results (Cardno 2014), the development footprint has been modified to minimise the potential for impact on mangroves. The original design reduced the channel width in the salt flats to the near the entrance of Pretty Pool Creek and resulted in increases to the current magnitude flowing through the mangroves under extreme conditions. Three different design options were examined and Option 2 was considered optimal for the channel width, and selected as the revised development footprint for further analysis.

For typical conditions, model results for the revised development footprint (Design Case) showed very little change in inundation level or current speed within the mangroves. Slight increases in both inundation level and current speed were predicted over the salt flats, which would tend to have a positive effect on mangrove condition, particularly at the seaward margin of the salt flats.

During extreme events, current speeds are likely to be sufficiently high to result in the erosion of sediments from the salt flats. This is predicted to occur for the existing conditions (Base Case), and the presence of the development (Design Case) increases the speed of currents and also therefore increases the likelihood of mobilisation of sediments. The Athol Design Case results in further increases in current speed, particularly in the salt flats to the west of the mangroves. This may result in redistribution of the mangrove habitat, however given the frequency of these events it is likely that other direct impacts (i.e. wind) will likely be more significant than the alterations to the hydrodynamic regime due to the development

The replacement of the very high salinity salt marsh (which limits the mangroves) with development fill will likely concentrate the fresh water input from the local catchment to the periphery of the developed area. This concentration of freshwater input may provide favourable conditions for the mangroves, with colonisation occurring in areas adjacent to the development that previously were limited due to high salinity.

Assessment of the stability of the Pretty Pool Creek entrance due to the effects of storm bite, historical trends and sea level rise has been undertaken. On the basis of the assumptions and assessment undertaken here it is concluded that this area is likely to remain relatively stable over the design life. However it is recommended that a geotechnical investigation be performed to establish the presence and depth of bedrock in this area to add additional confidence to this conclusion. Based on the hydrodynamic modelling undertaken it is unlikely that the development will result in hydrologic changes that effect the morphology of the entrance under ambient conditions, however under extreme conditions there may be some different morphological response of the creek and entrance due to the altered flow paths predicted.

On the basis of this preliminary assessment, the Stables development alone is not anticipated to have significant negative impacts on the mangroves. With the addition of the Athol Street development, higher levels of inundation are observed by water flowing over Cooke Point Drive. This road could be raised as part of the development process if this predicted increase of 15-20 cm in inundation is undesired.

6.1 Implications for Development

The development footprints have been modelled assuming the full Design footprint is to be used as residential development for a planning timeframe of 100 years. That is, it was assumed that the full footprint will be filled to a level of +6.6 m AHD and the area protected by a bund wall of suitable rock design for stability. This meets the SPP2.6 inundation criteria for the 100-year planning timeframe – above the 500-year ARI water level. As per the erosion investigation summarised in Section 5.6, the site is also landward of the erosion hazard line for the area, so the erosion hazard criteria of SPP2.6 for the 100-year planning timeframe are also met.

Should short term development be preferred in any portion of the Design footprint, the requirements of the site can be less demanding than a fully filled site. For example, if shorter design life, say 30 years, accommodation options are desired, these may be located within the Design development footprint without bunding, as long as the finished floor level (FFL) of any permanent structures are raised above the corresponding planning timeframe inundation level with associated localised scour protection. Plans for a managed retreat should be put in place during the lifetime of the development, to ensure appropriate use of the land occurs in line with climate change.

It is understood that a concept plan for a caravan park in the eastern portion of the Design footprint was put forward in 2013. This had caravan sites located within the 3.5 m AHD to 5 m AHD contours, with the more permanent structures located above the 5.5 m AHD contour. The Port Hedland Coastal Vulnerability Study (Cardno 2011) calculated the return period water levels for East Port Hedland. These are presented for the present and 2110 climate scenarios in Table 6-1.

From this table, it is apparent that the caravan sites would be inundated during the 2-year to 20-year ARI events. The permanent structures would only be inundated under the present day 500-year ARI event. Therefore, the caravan park site is probably suitable for a planning timeframe of roughly 50-years, however the caravan sites would likely be inundated several times during this timeframe.

Any short-term development proposed within the Design footprint should take these levels into consideration.

Table 6-1 Design Peak Total Still Water Level (TSWL), excluding wave setup, for East Port Hedland (Cardno, 2011)

ARI (years)	Peak TSWL (m AHD) Present Climate Scenario	Peak TSWL (m AHD) 2110 Climate Scenario
2	3.5	4.4
10	4.0	4.9
20	4.1	5.0
50	4.4	5.3
100	5.0	5.9
200	5.1	6.0
500	5.6	6.6

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APPENDIX B
GEOTECHNICAL INVESTIGATION REPORT

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JDSi Consulting Engineers

Report on Geotechnical Investigation

Stage 3, The Stables, East Port Hedland

30 September 2014

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Report on Geotechnical Investigation

Prepared for
JDSi Consulting Engineers

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For and on behalf of Coffey

Richard Moyle
Principal Geotechnical Engineer

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1. Introduction

This report presents the results of the geotechnical investigation carried out by Coffey Geotechnics Pty Ltd (Coffey) for JDSi Consulting Engineers (JDSi) associated with the Stables Development, East Port Hedland.

This work was commissioned by Glenn Coffey of JDSi on 13 August 2014 via a purchase order dated 25 July 2014 (Ref. 00000759).

2. Proposed development

We understand that the subject site is located 4km east of the Port Hedland Townsite and occupies approximately 28 hectares over three lots (Lot 5770, Lot 556 and Lot 300). The site is bound by Styles Road to the south, Cooke Point Drive to the west and the existing Pretty Pool residential subdivision to the east. To the north is Pretty Pool.

We understand that the part of the subject site is being leased to the Port Hedland Pony Club, which expires in December 2018. LandCorp are looking to rezone the subject site for residential development.

3. Objectives

The objectives of the geotechnical investigation were to complete Phase 2 works as outlined in our proposal (Ref. GEOTPERT10160AA-AA) and to provide key input in to the following activities:

- Conduct geotechnical and ASS field and laboratory testing and sampling, as appropriate, to satisfy the objectives of the investigation;
- Interpret laboratory and in situ test results to assess engineering soil parameters, as relevant to the development;
- Provide a preliminary assessment of the permeability of the materials;
- Prepare geotechnical reports as appropriate; and
- Provide a geotechnical report outlining the suitability of the site for development. This would also include a discussion of the key geotechnical risks and opportunities for the development.

4. Information supplied by JDSi

You have provided us with the following information:

- East Port Hedland Concept Plan Report (August 2012);
- East Port Hedland Geotechnical Reconnaissance;
- Phase 1 East Port Hedland Urban Water Advice;
- Port Hedland Coastal Vulnerability Study;
- Preliminary Environmental Assessment Report;
- Preliminary Noise Assessment;
- Stage 3 East Port Hedland – Request for Services;
- KMZ files for aboriginal site data and Pretty Pool site;

- LiDAR files covering project area; and
- Aerial Video footage of the site.

5. Fieldwork

5.1. General

Fieldwork was carried out on 11 September 2014 in the full time presence of personnel from Coffey. Test Pit locations were measured by handheld GPS relative to MGA94 and elevations were assessed using LiDAR contour plans provided.

Access to the site was from Styles Road and Sheridan Road. Access within the site was typically along existing unsealed tracks and was limited in in some area areas due to presence of soft estuarine deposits and spring tides. Access into the Stables area was limited due to presence of livestock. Trafficability at the time of fieldwork was suitable for 4 wheel drive and an excavator. Weather conditions were clear and fine.

Approximate investigation locations and general site photograph locations and directions are shown on Figure 1. General site photographs are shown in Figure 5 to 20.

5.2. Test pit excavations

A total of 16 test pits (TP1 to TP16) were excavated by a 5 tonne rubber tracked excavator to depths varying from 0.05m to 2.6m below the existing ground surface. Details of the test pit locations, including coordinates and elevations (based on client supplied LiDAR plan), are shown in Table 1.

Table 1 – Test Pit Coordinates and Elevations

Test Number	Easting (50 MGA 1994)	Northing (50 MGA 1994)	Elevation (mAHD)
TP1	670603	7752904	3.9
TP2	670569	7752944	2.9
TP3	670450	7752947	3.2
TP4	670463	7752984	2.6
TP5	670329	7753030	2.6
TP6	670218	7753068	2.4
TP7	670222	7753014	3.2
TP8	670184	7753019	3.1
TP9	670047	7753073	3.0
TP10	669953	7753102	3.0
TP11	670152	7753028	3.1
TP12	670328	7752994	3.7
TP13	670961	7752994	6.2
TP14	671155	7752923	3.8
TP15	671137	7752922	3.4
TP16	670703	7753138	2.8

The records of the test pit logs showing the major strata that were intersected, the depths at which the samples were taken, in-situ tests carried out, and the results of these tests, together with photographs and Explanation Sheets defining the terms used, are presented in Appendix A.

5.3. Hand probes

Due to the soft nature of the estuarine deposits and the spring tides, access to areas containing estuarine muds using an excavator was limited. In order to assess the thickness of estuarine deposits overlying the limestone hand probes were used to push through the soft estuarine deposits. The depth to limestone as assessed using the hand probes is illustrates on Figure 1.

5.4. Infiltration testing

Two falling head permeability tests, P1 and P2, were carried out across the site, as shown on Figure 1. Tests were completed within the upper 0.5m from ground surface level. The recommended design permeability value is discussed in Section 8.4.

6. Laboratory testing

Laboratory testing was carried out in accordance with the general requirements of AS 1289 by Coffey Information, a NATA registered soils laboratory.

A summary of the results are presented in Table 2. Test certificates are attached in Appendix B.

Table 2 – Laboratory Testing Results

Test Location	Depth (m)	Particle Size Distribution			Acid Sulfate Soil Results
		% Gravel	% Sand	% Fines	
P1	0.5	4	92	4	-
P2	0.5	1	94	5	-
TP2	0.1	-	-	-	Awaiting Results
TP2	0.4	-	-	-	Awaiting Results
TP4	0.5	-	-	-	Awaiting Results
TP4	1.0	-	-	-	Awaiting Results
TP5	1.4	-	-	-	Awaiting Results
TP6	1.3	-	-	-	Awaiting Results

We are awaiting the laboratory results of the Acid sulfate soils testing. Once results are received an assessment will be provided as an addendum to this report.

7. Site conditions

7.1. Surface conditions

The site occupies an irregular shaped area predominately bound by Styles Road to the south, Cooke Point Drive to the west and the existing Pretty Pool residential subdivision to the east and tidal flats to the north.

The site is approximately 28 hectares over three lots (Lot 5770, Lot 556 and Lot 300) and is situated on topography with ground elevations ranging from 8m AHD to about 2.4m. Lot 5770 is currently leased to the Port Hedland Pony Club for horse stable and pony club activities.

Vegetation predominantly comprises low lying shrubs, small trees grass and mangroves. A number of unsealed access track tracks exist across the site.

7.2. Subsurface conditions

The 1:50,000 Geology Series map (Port Hedland sheet) indicates that the subsurface profile comprises dune limestone, mud and silt, older dune shelly sand and mobile dunes as shown on Figure 2.

Based on the field investigation and the above map the site has a generalised subsurface profile presented in Table 3.

Table 3 – Generalised Subsurface Profile

Area	Unit	Description	Remarks
A	Dune Sand	Sand; Fine to coarse grained sand with trace silt gravels and shell, loose to medium dense	Previous investigations in the area indicate the dune sand is underlain by Estuarine muds as illustrated in TP15, or by limestone.
B	Estuarine Deposits	Sandy Clay/Clay: low to high plasticity, grey and brown, generally very soft to soft, stiff to hard in some areas	Typically underlain by Limestone, and overlain by Dune Sand
C	Limestone	Pale brown/yellow, well to very well cemented, low to high strength, occasional voids	Calcarnite and Calcisilite likely to be underlain by red brown silty Sand (Red Beds). Typically overlain by Estuarine deposits and Dune Sands. Overlain by thin layer of clay/sand in some areas.

Inferred boundaries delineating Areas A, B and C within the site are illustrated on Figure 3. In the western portion of the site a shallow basin of estuarine deposits overlies the limestone as illustrated by the hatched area on Figure 3.

Figure 1 includes the location of sub-surface sections through the site. The sections are presented on Drawings 1 to 4 and show the interpolated sub-surface profile across the site.

It should be noted Cooke Point Drive boarders the western boundary of the site and it is likely that fill material associated with the construction of the road exists in this area.

7.3. Groundwater

Groundwater levels measured during the course of the investigation are presented in Table 4.

Table 4 – Groundwater Elevations

Test Number	Depth to Groundwater (m)	Approx. Surface Level (mAHD)	Approx. Level of Groundwater (mAHD)
TP1	Not Encountered	3.9	Not Encountered
TP2	Not Encountered	2.9	Not Encountered
TP3	Not Encountered	3.2	Not Encountered
TP4	0.9	2.6	1.7
TP5	1.4	2.6	1.2
TP6	1.4	2.4	1
TP7	Not Encountered	3.2	Not Encountered
TP8	Not Encountered	3.1	Not Encountered
TP9	0.9	3.0	2.1
TP10	1.1	3.0	2.9
TP11	0.4	3.1	2.7
TP12	Not Encountered	3.7	Not Encountered
TP13	Not Encountered	6.2	Not Encountered
TP14	Not Encountered	3.8	Not Encountered
TP15	1.9	3.4	1.5
TP16	1.9	2.8	0.9

It should be noted that groundwater levels on a particular site are influenced by several factors including:

- Regional groundwater levels;
- Local Geology;
- Rainfall;
- Tides;
- Local and Regional Drainage;
- Changes in land use;
- Groundwater extraction.

Rainfall has a major effect on groundwater levels, particularly from November through to March (i.e. during “the wet season”). The hydrology and stormwater diversion for the area will need to be considered, but fall outside the scope of this study. The process of urbanisation can affect groundwater levels. Road paving and house construction removes a portion of the soil surface from which evaporation can take place. Whether or not roof runoff is piped off site or returned to the soil via soak wells or directly off the roof will also have an effect.

Groundwater levels recorded in July 2013 in the vicinity of the site indicate that groundwater levels would be approximately R.L. 2.5m AHD to R.L. 3m AHD. Generally this coincides with a groundwater level perched on the Estuarine Muds. As such, there is likely to be a significant range in groundwater level, particularly after a rain event or “the wet season”.

Tides are likely to influence the groundwater levels at the site. Based on Foulsham (2010), the following tidal data is relevant:

- Highest Astronomical Tide (HAT): 3.59m AHD;
- Mean high water spring tide: 2.79m AHD;
- Mean Sea Level: 0.0m AHD;
- Mean low water spring tide: -3.01m AHD
- Lowest Astronomical Tide (LAT): -3.92m AHD;
- 1 in 50 year storm surge level: 4.9m AHD; and
- 1 in 100 year storm surge level: 5.4m AHD.

8. Recommendations

8.1. General

It should be noted that the ground encountered by the test pits represent the ground conditions at the location where the tests have been undertaken and as such are an extremely small proportion of the site to be developed. Accordingly, variations to the ground conditions are likely and allowance should be made for variability in the design and construction budgets.

Whilst, to the best of our knowledge, the information contained in this report is accurate at the date of issue, ground conditions including groundwater levels can change in a limited time or due to seasonal fluctuations. For example fill could be added to a site or surface materials removed from a site that will change the thickness of surface materials and depth to the underlying materials. The potential for change in ground conditions should be recognised particularly if this report is used after a protracted delay.

It is also recommended that any plans and/or specifications prepared which relate to the content of this report or amendments to original plans and specifications be reviewed by Coffey to verify that the intent of the recommendations contained in this report are properly reflected in the design.

8.2. Acid sulfate soils

Figure 6 of the Western Australia Planning Commission (WAPC) Planning Bulletin 64 – shows the site to be located in an area of low to high risk of ASS occurring within 3m of natural soil surface (or deeper).

We are awaiting the laboratory results of the acid sulfate soils testing. Once results are received they will an assessment will be provided as an addendum to this report.

8.3. Site classification

Australian Standard AS2870-2011 provides a system of site classification for residential slabs and footing design as follows:

Table 5 – General Definition of Site Classes

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which may experience moderate ground movement from moisture changes
H1	Highly reactive clay site, which may experience high ground movement from moisture changes
H2	Highly reactive clay site, which may experience very high ground movement from moisture changes
E	Extremely reactive sites, which may experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include: Soft soils, such as soft clays or silts or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Port Hedland is located to the north of the Tropic of Capricorn and as such experiences a dry climate. Australian Standard AS 2870, *Residential Slabs and Footings* adopts a depth of design suction change (H_s) of 4m for site classification foundation design.

The standard recommends that for sites with deep-seated moisture changes characteristic of dry climates and corresponding to H_s equal to or greater than 3m, the classification shall be M-D, H1-D, H2-D or E-D as appropriate.

As such a minimum foundation design of **Class M-D** should be adopted for construction within Area A on Figure 1.

In the case of the Area B a classification of **Class P** would be appropriate given their very low bearing capacity and excessive settlements. In such cases foundations are to be designed based on engineering principles. As granular fill will be required to bring the site to design levels the standard allows for the reclassified, in accordance with engineering principles, to a classification lower than Class P and the adoption of the standard footing designs.

In the case of the Area C a classification of **Class A** would be appropriate.

These classifications are judged to be appropriate for the site provided that the recommendations contained in this report are adopted.

It is anticipated that the Area A and B could potentially be classified as **Class A-D**, as fill may be required to bring ground level up to design level and as such the maximum depth to seasonal moisture variation may be located within non-cohesive soil, resulting in negligible potential for surface movement due to seasonal variation.

8.4. Soak wells and sumps

The natural sand soils identified in the TP13 and in Area A are considered to be free draining, whilst the estuarine deposits and Limestone are not.

The process of urbanisation can lead to an increase in the rate of runoff from rainfall events. Rainfall that would otherwise infiltrate and move slowly through the soil to streams, lakes and rivers, when directed into gullies and pipes will move rapidly and create adverse environmental effects. The use of soak wells is likely to slow the movement of water there is likely to be compatible with the principles of Water Sensitive Urban Design. However the use of soak wells can have adverse effects on footing performance and/or amenity of a residential development.

In Port Hedland, it is common to have houses without gutters and rainfall runs directly off the roof onto the ground (or a designed gravel drainage layer). Therefore, the design of a surface water disposal, including rainfall runoff and collection, becomes an issue rather than groundwater disposal.

Should soak wells be considered, then they need to be designed based on a perched water table sitting on or above the Estuarine Muds. As a guide, for the purpose of soak well/ sump design, the water table is regarded as shallow if the depth to water table below the base of the soak well/sump is less than the square root of the base area. For a typical domestic soak well with a nominal height of 600mm and a nominal diameter of 600m, a depth to the winter water level (from finished lot level) of about 1.2m to 1.5m is required for soak wells to operate efficiently.

The use of a mixed system involving provision of sufficient storage volume in soak wells for short duration high intensity rainfall events and provision of controlled over flow for long duration rainfall events can be considered.

Based on our experience, analysis of laboratory results of the sand soils within Area A and to allow for silting effects a permeability value of 5m per day should be used in the design of soakwells and basins with in Area A. Consideration should be given to grading of development towards road reserves and low lying areas in order to direct runoff to these areas.

8.5. Foundations

8.5.1. Shallow foundations

We recommend an allowable bearing capacity of 150kPa provided the following:

- Area A - ground surface is prepared in accordance with the recommendations provided in Section 9.2.
- Area B - ground improvement recommendations provided in Section 10 are undertaken, including a detailed design phase of the works,
- Area C - ground surface is prepared in accordance with the recommendations provided in Section 9.4.

The following should be noted about the shallow foundations:

- Where foundations are founded in engineered fill material that that material should be compacted to accordance with Table 7;
- There should be at least 2m of fill between the top of the estuarine deposits layer and the bottom of the foundation; and
- The allowable bearing pressure for the footings should be limited to no more than 150kPa.
- Allowable bearing pressures are based on the following assumptions:
  - Load eccentricity is less than 10% of footing width;
  - Load inclination (H/V) is less than 10%;
  - The ground surface is horizontal; and
  - There is no interaction between adjacent footings.

Where the above recommendations cannot be achieved, then the ground improvement works recommended in Section 10 may have to be refined. These adjustments could be made during the detailed design phase of the works.

8.5.2. Deep foundations

Deep foundations may be considered for the structures in lieu of carrying out significant ground improvement works. Pile design parameters are provided in the following sub-sections.

It should be noted that if piling is considered for the structures then adequate allowance has to be made for differential settlement between piled and non-piled sections of the site. The differential settlements will typically be the greatest at connections to houses and as such services and connections to houses will need to be designed accordingly.

Pile Types

It is recommended that bored piles be considered for use at the site. Bored piles have the advantage of relatively low vibrations during the installation process and can be implemented with high quality control procedures.

The bored piles would also be effective in assessing that the founding stratum (Limestone) has been intersected and an adequate embedment has been achieved.

Estimated Safe Working Loads

Safe working loads for a range of pile diameters have been estimated using the recommendations provided in Fleming *et al* (1992) and are presented in Table 6. The estimation of the pile capacity is assuming on end bearing piles founding on the Limestone.

Table 6 - Safe Working Loads for Pile Under Axial Compression

Pile Diameter (mm)	Safe Working Load (kN)
300	45
400	85
500	130
600	185

The safe working loads have been estimated assuming a global factor of safety of three. Should piling be considered further for the site, further analysis will need to be undertaken to confirm the pile capacity in accordance with AS 2159 – 2009, taking into account a geotechnical strength reduction factor (which is based on the number and type of pile testing undertaken, and the average risk rating for the development).

8.6. Flexible pavements

8.6.1. Subgrade CBR

In Area A that is underlain by Dunal Deposits, a preliminary design subgrade California Bearing Ratio (CBR) of 10% is recommended. The adoption of this design value is contingent upon strict compliance with the site preparation recommendations given in Section 9.2

Where excess limestone in from Area C won on site, this material may be crush and reused as structural fill. A preliminary design subgrade CBR of 30% is recommended for crushed limestone material.

Where areas are to be built up to the design level using imported fill materials, the design CBR value will be dependent upon the material used in the filling process. Based on our experiences in the Port Hedland area, the range in CBR values could be between 5% and 10%. As part of the quality assurance testing programme of the importation of fill, CBR testing should be undertaken of the near surface materials to confirm the design CBR values.

The estuarine muds are considered to form poor quality subgrade. For design of temporary haul roads to facilitate construction, a subgrade CBR of 2% is recommended.

8.6.2. Pavement design

For medium and heavily trafficked public roads with an asphalt surface, pavement thickness design should be/have been based on Austroads (2012) Guide to Pavement Technology Part 2: Pavement Structural Design.

For lightly trafficked public roads, pavement thickness design should be/have been based on the Austroads (2006) Supplement to Austroads Pavement Design Guide. For public roads with a sprayed seal surface, pavement thickness design may be/has been based on the empirical design chart in Main Roads WA Engineering Road Note 9 (2013).

In intersections and tight turning areas, asphalt surfacing must be provided at intersections and tight turning areas.

Based on our experiences with pavements for the Pretty Pool Stage 1 Development, the recommended pavement composition is:

- Surface 30mm thickness of dense graded asphalt with 10mm aggregate
  - tack coat
  - 2 coat emulsion primer seal
  - prime
- Base Course 150mm Crushed Rock Base / Lateritic Gravel
- Sub Base 150mm Crushed Lateritic Gravel / Pindan Sand

8.6.3. Pavement materials

Pavement materials for public roads should conform to the “Guide to the Selection and Use of Naturally Occurring Materials as Base and Sub Base” jointly published by Main Roads Western Australia and Australian Geomechanics Society (2003).

It must be understood that some cracking is normal if self-stabilising materials (lateritic gravel, ferricrete, calcrete), cement stabilised material or material with a significant linear shrinkage is used. Cracking is more visually apparent when asphalt surfacing is used compared to a sprayed seal. Cracking (which may originate in the base course) is more apparent when asphalt manufactured using laterite or ferricrete (ie red asphalt) is used as the surfacing.

The method for construction must comply with current MRWA specifications.

8.7. Earthquake parameters and liquefaction

Based on the site investigation and AS 1170.4 – 2007 Structural Design Actions – Part 4: Earthquake actions in Australia, the site subsoil class is estimated as follows:

- Class C_e (shallow soil site) in Area A (Dune Sand);
- Class D_e (Deep of soft soil site) in Area B (Estuarine deposits); and
- Class B_e (rock site) in Area C (Limestone).

The Hazard Factor (Z), as defined in AS 1170.4 – 2007, for Port Hedland is 0.12.

Soil liquefaction tends to occur in saturated, loose sandy soils that are subject to high seismic stress (cyclic). As a result, the sandy soils lose strength and stiffness and may undergo significant settlement or displacement. In the Dune Deposits, there is potential for isolated loose zones that may be susceptible to liquefaction. The result of an isolated loose zone liquefying would result in surface settlements.

It should be noted that identification of such layers through target geotechnical investigations would be difficult. Therefore, it is recommended that the structures be designed for liquefaction using the above parameters, and should an earthquake occur, then a maintenance regime be adopted for any damages that occur.

9. Earthworks

9.1. General

Earthworks should be carried out in accordance with the principles set out in AS3798-2007 Earthworks for Residential and Commercial Developments.

9.2. Area A (dune sands)

All organic materials (and any uncontrolled fill) should be stripped and stockpiled. This would include the stripping of surface vegetation.

The organic material is not suitable for use as structural filling. It is only suitable for landscaping purposes.

It should be noted that ground conditions and particularly groundwater levels may vary with the seasons. As such, site preparation procedures may differ from the above if development proceeds during the wet season.

After the site has been stripped to the satisfaction of the Supervising Engineer, the Dunal Deposits should be proof compacted using a heavy, self-propelled, smooth drum vibrating roller, capable of operating in variable frequency modes. A Dynapac CA250D, or equivalent, is recommended (subject to the protection of adjacent buildings from damaging ground vibrations).

For sand with a fines content of less than 8%, a smooth drum roller should be used. For material with fines content exceeding 8% a pad foot roller will be required.

The following proof compaction procedure is recommended:

- The entire site should be given a minimum of 4 passes with the roller operating in high amplitude mode. A pass should include a minimum overlap of 20%.
- The site should then be given an additional minimum of 4 passes with the roller operating in low amplitude mode.
- All weak areas that deform excessively under rolling, should be removed and replaced with suitable material.
- On completion of vibratory rolling, 2 passes of the site should be made with the roller operating in a static mode. This will compact the sands in the upper 300mm that were disturbed by cyclic mobility.

It is recommended that the proof compaction be monitored by an Engineer experienced in earthworks.

It should be noted that this area may have deeper underlying Estuarine muds and as such may require further geotechnical investigation and ground improvement options, as discussed in Sections 10 and 11.2.

9.3. Area B (estuarine deposits)

All organic materials (and any uncontrolled fill) should be stripped and stockpiled. This would include the stripping of surface vegetation.

Stripping of vegetation over the estuarine deposits may be problematic, in particular during the wet season or during periods of high tide. As such, engineering judgement should be made regarding the benefit of stripping topsoil versus the difficulty in accessing the softer areas.

Consideration could be given to the placement of a geosynthetic layer over the estuarine deposits to assist in providing a trafficable surface. The geosynthetic layer would also provide tensile strength within the fill and act as a separation layer between the Estuarine deposits and the placed fill materials. At this preliminary stage, a product with a tensile strength of >20kN/m at 2% strain should be used.

In areas where there is estuarine deposits, proof rolling is not recommended.

It is anticipated that the estuarine deposits may be excavated to depth of approximately 2m to expose limestone beneath, prior to back fill with structural fill. The intersection of the estuarine deposits with the limestone at ground surface (0m thickness contour) and also the inferred 2m thickness contour are shown on Figure 3.

Alternatively or in addition to earthworks above, Coffey recommend deep foundations or ground improvement options in areas of Estuarine deposits. These options are outlined in Sections 8.5.2 and 10.

It should be noted that although it common practice to remove organic material; it may be beneficial to leave the mangrove root mat in place in order to act as a natural raft and improve trafficability in the area. This method could be adopted provided it is factored into the end design.

9.4. Area C (limestone)

All organic materials (and any uncontrolled fill) should be stripped and stockpiled. This would include the stripping of surface vegetation. It is anticipated that rock breaking or ripping of the limestone will be required where lowering of existing ground surface is necessary, and importing of structural fill material may be required where raising of ground level is required.

9.5. Temporary slopes during earthworks

Excavated slopes should be constructed not steeper than IV:3H. Generally the temporary slopes should have the following:

- slope drainage consisting of catch drains; and,
- erosion protection should be provided.

9.6. Suitability of site materials for use as fill

Based on limited geotechnical investigation, the dunal deposits and limestone excavated from site may be used as structural fill. However, the material or additional material imported on to the site should be the subject of an investigation and laboratory testing regime to confirm the grading and compaction characteristics of the material.

It is unlikely that the Estuarine Materials will be suitable for use as structural fill.

Topsoil may be used as fill in landscape areas but should not be used as structural fill.

9.7. Compaction requirements

Earthworks should be compacted to achieve the density requirements set out in Table 7.

Table 7 – Compaction Requirements

Item	Application	Compaction Criteria		
		Minimum density ratio (Cohesive soils)	Minimum density index (Cohesionless soils)	Minimum Dry Density Ratio (Perth sands)
1	Residential – lot fill, house sites	95% std	65%	95% mod
2	Commercial – fills to support minor loadings, including floor loadings of up to 20 kPa and isolated pad or strip footings to 100 kPa	98% std	70%	96% mod

10. Preliminary ground improvement options

10.1. General

At this stage, Coffey recommend three potential ground improvement options for development across the containing estuarine deposits.

The proposed options for site development include:

- Preloading with wick drains;
- Vibro-replacement columns (stone or sand columns); and
- Controlled modulus columns.

Preliminary design information regarding these options can be provided if required.

10.2. Preloading and wick drains

Preloading the area following installation of wick drains can be designed to accommodate 90% consolidation to be completed within significantly less time period than that of preloading alone, depending on the spacing and performance of the drains. When wick drains are introduced, rapid dissipation of pore pressure occurs leading to higher settlements in shorter time. However, the effects of smearing during installation, reduced permeability under increased stress and possibility of kinking are concerns associated with wick drains.

Wick drains have become a very common method to improve soft ground and have become routine.

10.3. Vibro-replacement

Vibro-replacement is a method of forming sand or stone columns within the Estuarine deposits; founding in the underlying Limestone layer. The use of stone or sand columns would need to be incorporated with a load transfer platform at (or near) the design finish ground level.

The stone or sand columns would act as a means of transferring the load of the development through the soil to a hard founding layer (similar to that of a pile) but also have the added benefit of assisting in the consolidation of the soft clayey material.

The economic viability of stone or sand columns is highly dependent upon the supply of suitable stone or sand; hence the availability of this resource would need to be assessed before considering this option further.

10.4. Controlled modulus columns with load transfer platform

Controlled Modulus Columns (CMC) with a Load Transfer Platform (LTP) is considered to be in the same class of deformable foundation systems (DFS) such as stone columns, dynamic replacement columns, etc. However, due to the advancement of installation technology, quality control aspects and speed of construction, this method is now considered bridging the gap between Rigid Deep Foundations such as, reinforced concrete columns, steel caissons, etc., and DFS. These DFS's utilise comparatively deformable inclusions made of granular materials, cement grouts etc., and supports the structural or other loads through a distribution mat without a structural connection between the columns and the distribution mat (i.e. load transfer platform (LTP)).

11. Geotechnical considerations

11.1. Issues

Based on the results of the investigation, our understanding of the development and our understanding of the geotechnical issues in the Port Hedland area, we believe the main geotechnical issues are as follows:

- The Estuarine deposits will have a significant amount of settlement when fill material is placed over them to bring the site up to the design service level. The amount of settlement will need to be allowed for when initially constructing the site such that the ground surface level remains above the design level.

- Services within area B will be subject to a large amount of settlement. Considering the type of development being proposed, it is recommended that replacement of services be factored into a maintenance regime. This should include water, drainage, electricity, sewerage and gas.
- Sourcing of fill in Port Hedland is different to the Perth region, in that the fill often contains a large amount of fines. Potential sources of fill material should be identified as early as possible, and the supplier should provide classification testing to confirm the quality of the available material.
- Area A covered by Dune deposits may have deeper / underlying Estuarine deposits. As such, this area should be subject to a geotechnical investigation using boreholes to confirm the stratigraphy in this area.
- The use of soak wells should be designed bearing in mind the potential for a high groundwater level during the wet season perched on top of the Estuarine deposits. The soak well design should also be commensurate with the nature of any imported fill materials. However, issues associated with soak wells can be mitigated by not incorporating gutters into the housing design.

11.2. Recommendations for further geotechnical investigations

Based on the results of the initial investigation, we would suggest undertaking the following additional geotechnical investigation once a more detailed site layout has been established: This investigation could include, but is not limited to:

- Boreholes or Cone Penetration Test (CPT) up to 10m depth in the Dune Deposits;
- Additional test pits for the collection of samples for laboratory testing;
- Due to limited access to the stables and livestock within the, the area should be targeted for future geotechnical investigations;
- Laboratory testing;
- Permeability testing; and
- Reporting to address the above mentioned geotechnical issues.

For budgeting purposes, it is recommended that an allowance of say $50,000 be made for the additional investigation, laboratory and reporting works.

Important information about your Coffey Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions.

For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report*

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples.

These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

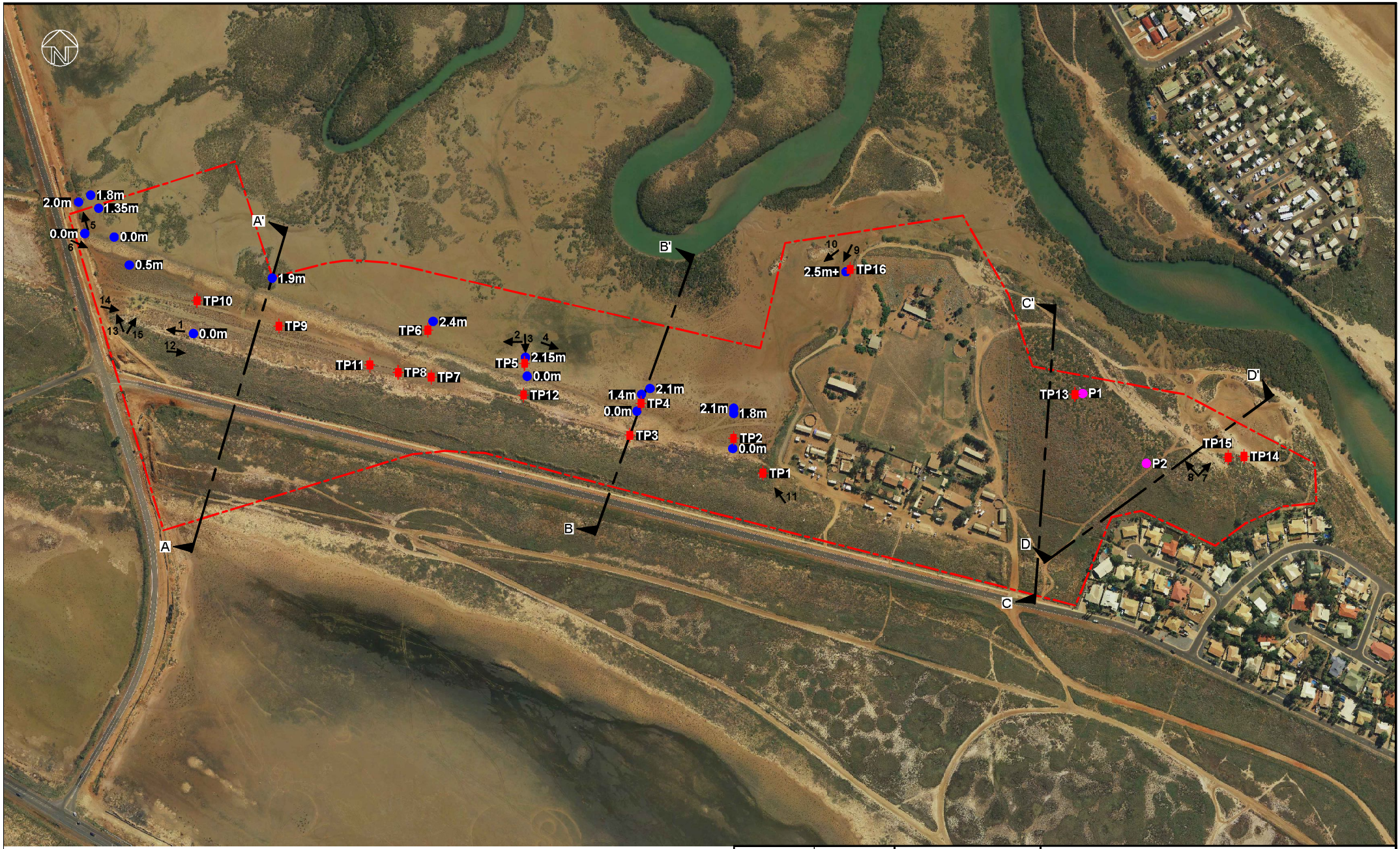
Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical information in Construction Contracts" published by the Institution of Engineers Australia, National headquarters, Canberra, 1987.

Figures



LEGEND

- TEST PIT LOCATION
- PERMEABILITY TEST LOCATION
- HAND PROBE TEST LOCATION
- PHOTO LOCATION AND DIRECTION
- SITE BOUNDARY
- SECTION LOCATION

0 40 80 120 160m

drawn	AMG
approved	RM
date	25/09/2014
scale	1:4000
original size	A3

coffey

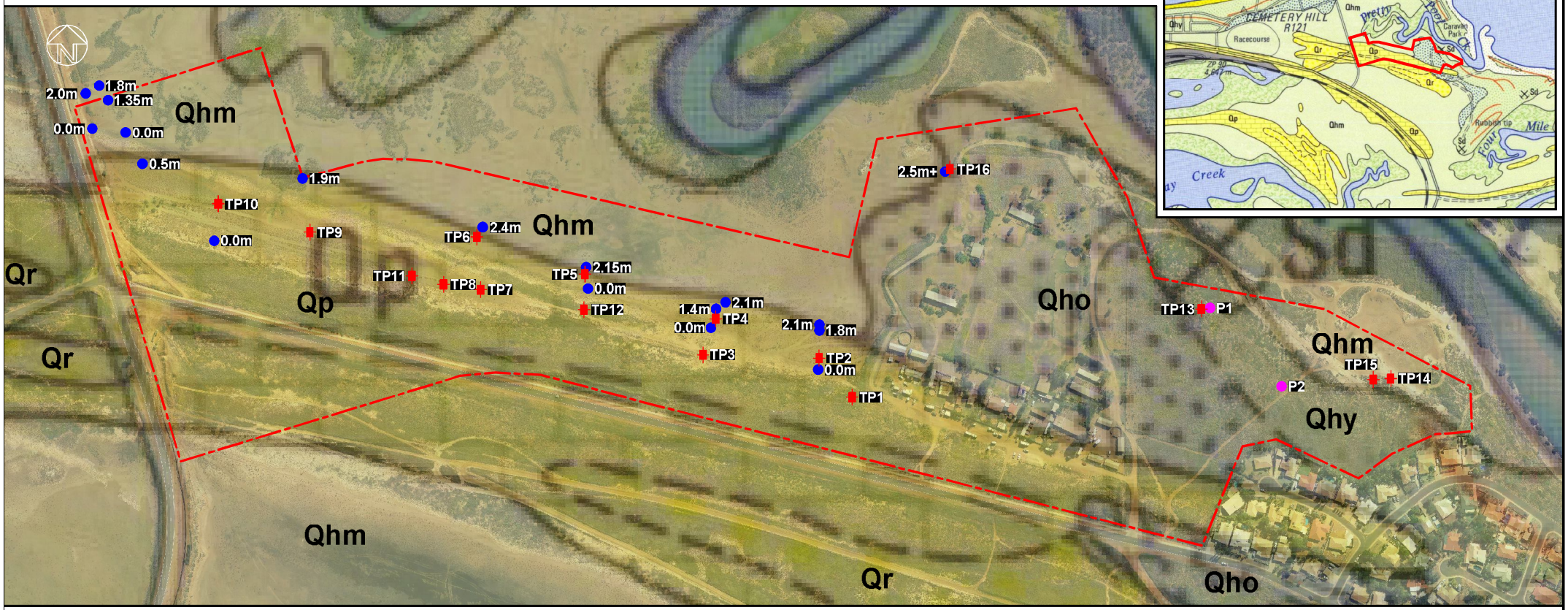
client: JDSi CONSULTING ENGINEERS

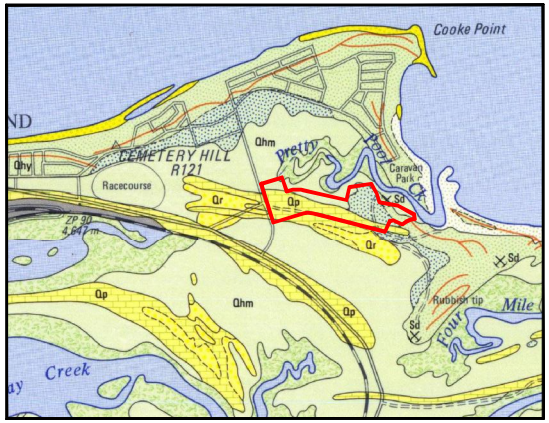
project: STAGE 3 - THE STABLES
EAST PORT HEDLAND, WA

title: TEST LOCATION AND SITE PLAN WITH AERIAL OVERLAY

project no: GEOTPER10160AA | fig no: FIGURE 1 | rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST LOCS AND SECTIONS.dwg





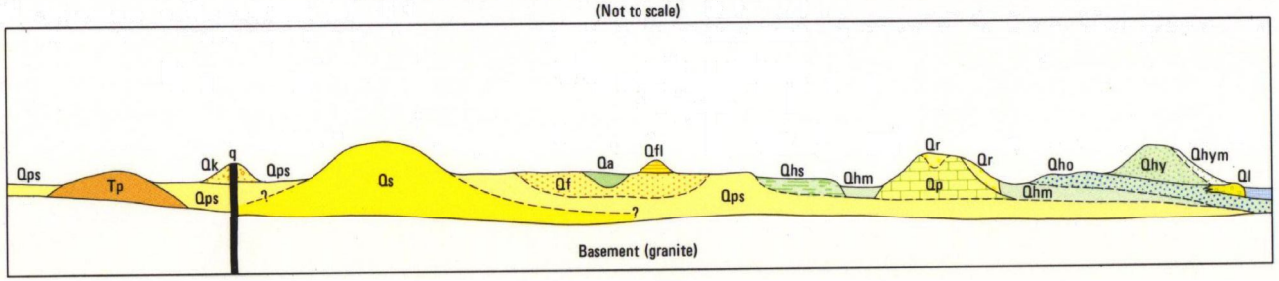
MAXIMUM THICKNESS (m)		
Qhm	5	Mangrove flats (tidal)
		Mud and silt
Qhym	2	Mobile dunes
Qhy	20	Younger beach and dune shelly sand
Qho	5	Older dune shelly sand
Qhs	3	Silty sand, red-brown, containing .....
Qr	3	Residual sand
Qp	? 20	Dune limestone
Ql	? 10	Lime-cemented beach conglomerate
Qfl	? 10	Levees
Qf	? 15	Flood-plain sediments
Qa	15	Alluvium, gravel, sand and silt
Qk	10	Colluvium, quartz scree
Qc	? 5	Clayey sand, abundant claypans
Qps	? 30	Silty sand, red-brown
Qs	? 40	High-level sands
Tp	20	POONDANOO FORMATION : laterite
q	30	Quartz; milky-white to grey

LEGEND

- TEST PIT LOCATION
- PERMEABILITY TEST LOCATION
- HAND PROBE TEST LOCATION
- SITE BOUNDARY

QUATERNARY SEDIMENT RELATIONSHIP DIAGRAM

(Not to scale)



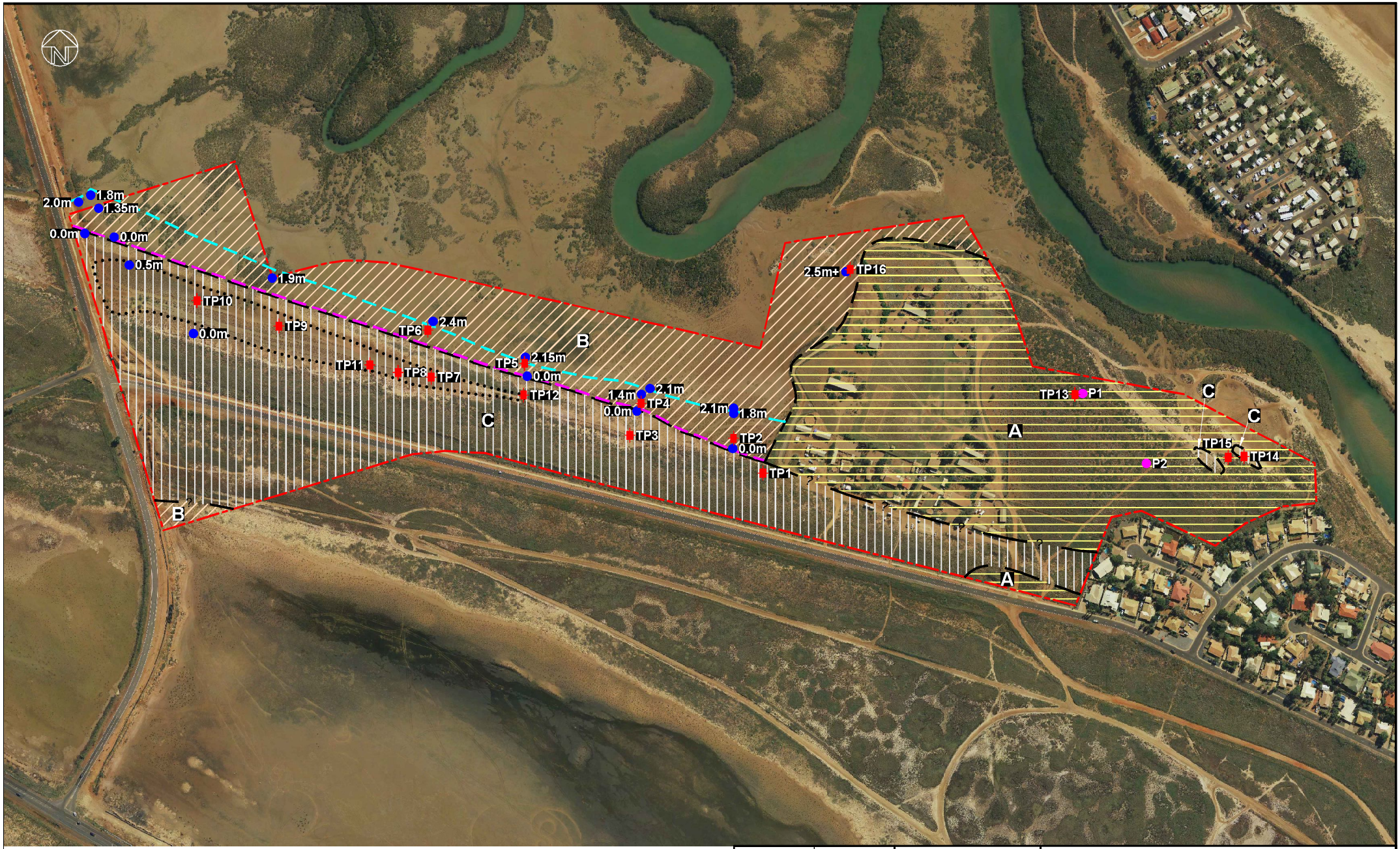
0 40 80 120 160m

drawn	AMG
approved	RM
date	25/09/2014
scale	1:4000
original size	A3

coffey

client:	JDSi CONSULTING ENGINEERS
project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA
title:	GEOLOGY AND AERIAL OVERLAY PLAN
project no: GEOTPER10160AA	fig no: FIGURE 2 rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST LOCS AND SECTIONS.dwg



LEGEND

- TEST PIT LOCATION
- PERMEABILITY TEST LOCATION
- HAND PROBE TEST LOCATION
- SITE BOUNDARY
- SHALLOW BASIN OF ESTUARINE DEPOSIT OVERLYING LIMESTONE
- AREA A - DUNE SAND
- AREA B - ESTUARINE DEPOSIT
- AREA C - LIMESTONE
- 2m ESTUARINE DEPOSIT THICKNESS CONTOUR
- 0m ESTUARINE DEPOSIT THICKNESS CONTOUR

0 40 80 120 160m

drawn	AMG
approved	RM
date	25/09/2014
scale	1:4000
original size	A3

coffey

client: JDSi CONSULTING ENGINEERS

project: STAGE 3 - THE STABLES
EAST PORT HEDLAND, WA

title: GEOLOGICAL AREA LOCATION PLAN

project no: GEOTPER10160AA | fig no: FIGURE 3 | rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST LOCS AND SECTIONS.dwg

LEGEND

- TEST PIT LOCATION
- PERMEABILITY TEST LOCATION
- HAND PROBE TEST LOCATION
- SITE BOUNDARY
- CONTOUR LINE

0 40 80 120 160m

drawn	AMG
approved	RM
date	25/09/2014
scale	1:4000
original size	A3

coffey

client: JDSi CONSULTING ENGINEERS

project: STAGE 3 - THE STABLES
EAST PORT HEDLAND, WA

title: SITE PLAN WITH TOPOGRAPHY OVERLAY PLAN

project no: GEOTPER10160AA | fig no: FIGURE 4 | rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST LOCS AND SECTIONS.dwg

PHOTOGRAPH 1

PHOTOGRAPH 2

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPHS**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 5**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - SITE PHOTOS.dwg

PHOTOGRAPH 3

PHOTOGRAPH 4

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPHS**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 6**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - SITE PHOTOS.dwg

PHOTOGRAPH 5

PHOTOGRAPH 6

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPHS**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 7**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - SITE PHOTOS.dwg

PHOTOGRAPH 7

PHOTOGRAPH 8

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPHS**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 8**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - SITE PHOTOS.dwg

PHOTOGRAPH 9

PHOTOGRAPH 10

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPHS**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 9**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - SITE PHOTOS.dwg

PHOTOGRAPH 11

PHOTOGRAPH 12

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPHS**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 10**	rev:

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PHOTOGRAPH 13

PHOTOGRAPH 14

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPHS**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 11**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - SITE PHOTOS.dwg

PHOTOGRAPH 15

NOTE: SEE FIGURE 1 FOR PHOTOGRAPH LOCATION AND DIRECTION

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**SITE PHOTOGRAPH**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **FIGURE 12**	rev:

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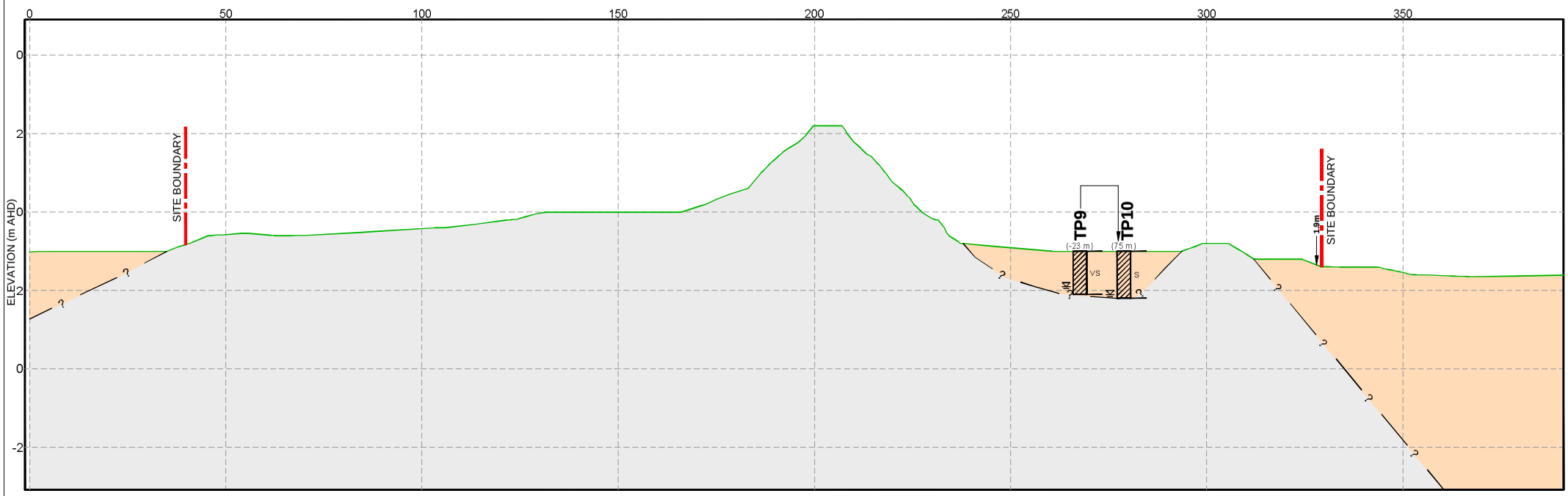
Drawings



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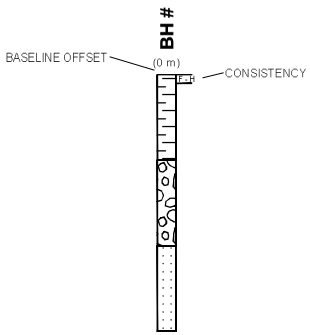


MATERIAL GRAPHIC

CLAY

LEGEND

- ESTUARINE DEPOSIT
- LIMESTONE
- INFERRED BOUNDARY
- NATURAL SURFACE

Geological boundaries are only known at the test site locations and have been inferred between the test sites. These geological boundaries have been provided to assist with the geological interpretation and should not be considered to represent actual boundaries that may vary from these lines.

H 1:1000

0 10 20 30 40 50m

V 1:100

0 1 2 3 4 5m

drawn	AMG
approved	RM
date	25/09/2014
scale	1:1000H 1:100V
original size	A3

coffey

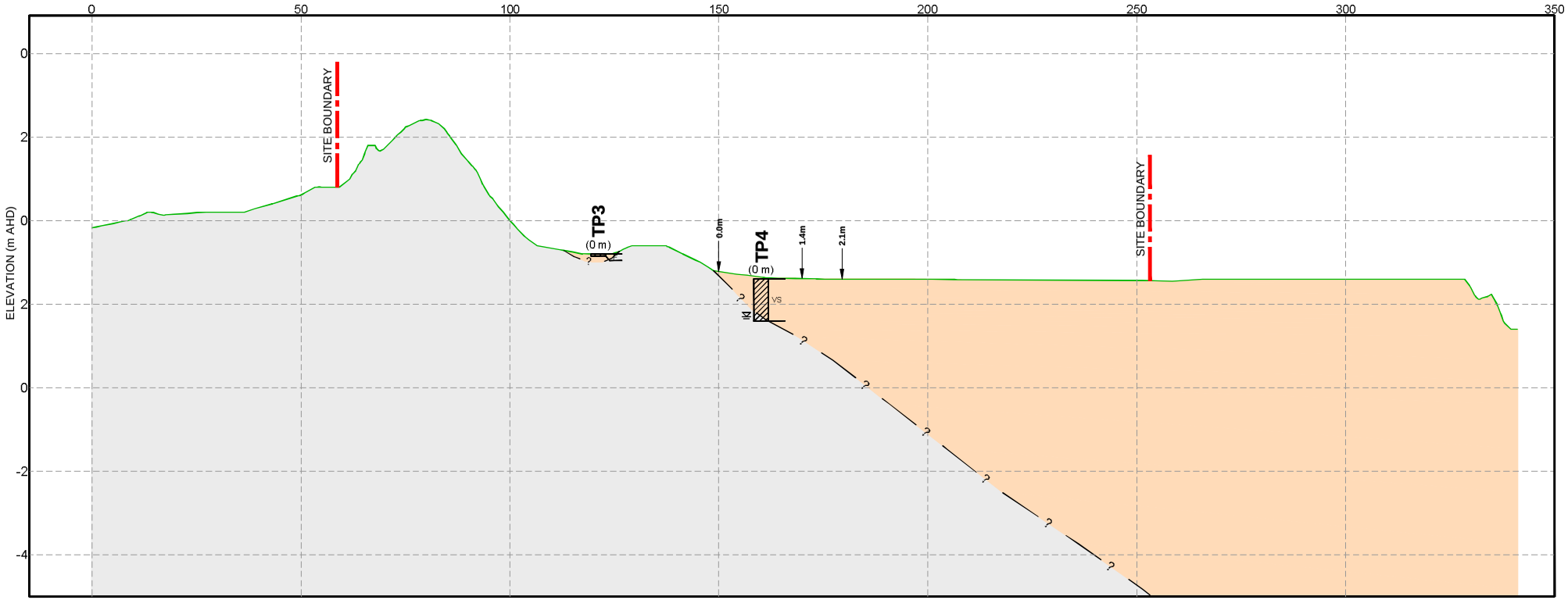
client: JDSi CONSULTING ENGINEERS

project: STAGE 3 - THE STABLES EAST PORT HEDLAND, WA

title: SUB-SURFACE SECTION A-A'

project no: GEOTPER10160AA | fig no: DRAWING 1 | rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST LOCS AND SECTIONS.dwg



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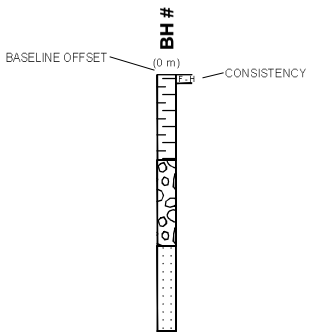


MATERIAL GRAPHIC

CLAY

LEGEND

- ESTUARINE DEPOSIT
- LIMESTONE
- INFERRED BOUNDARY
- NATURAL SURFACE

Geological boundaries are only known at the test site locations and have been inferred between the test sites. These geological boundaries have been provided to assist with the geological interpretation and should not be considered to represent actual boundaries that may vary from these lines.

H 1:1000

0 10 20 30 40 50m

V 1:100

0 1 2 3 4 5m

drawn	AMG
approved	RM
date	25/09/2014
scale	1:1000H 1:100V
original size	A3

coffey

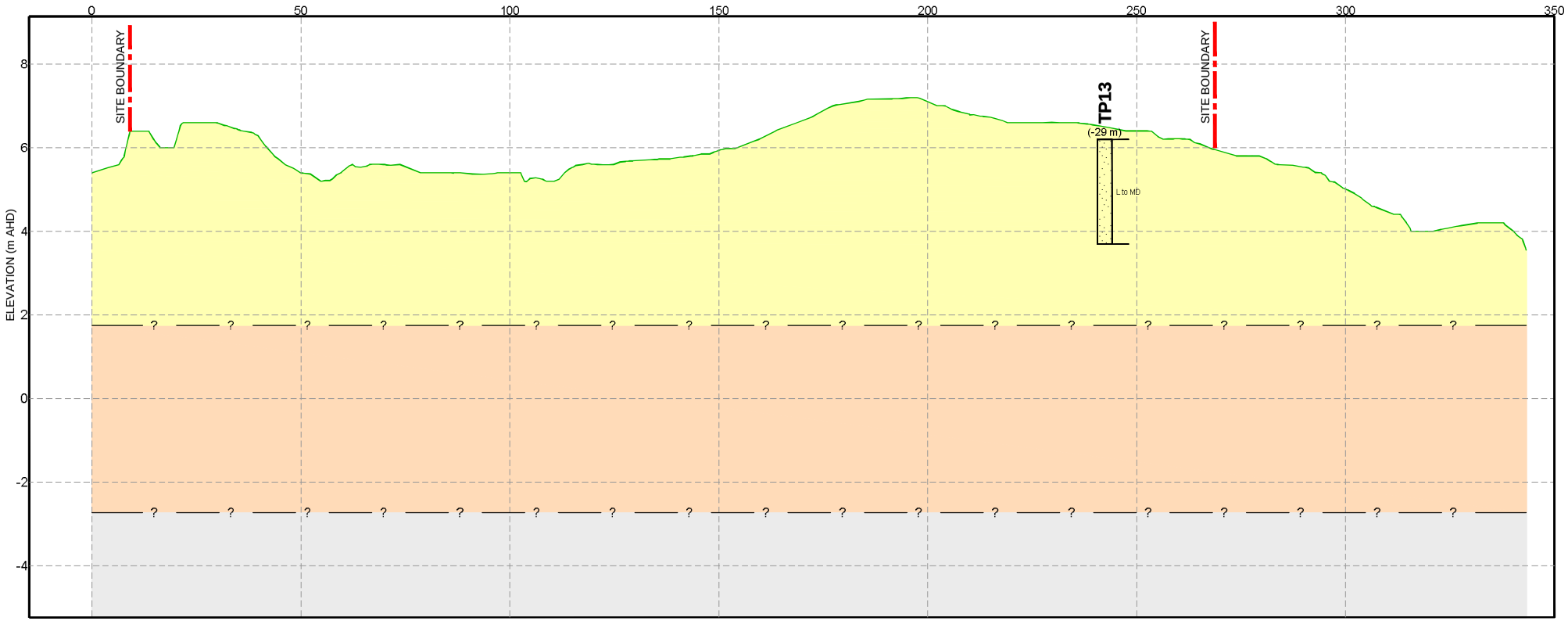
client: JDSi CONSULTING ENGINEERS

project: STAGE 3 - THE STABLES EAST PORT HEDLAND, WA

title: SUB-SURFACE SECTION B-B'

project no: GEOTPER10160AA | fig no: DRAWING 2 | rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST LOCS AND SECTIONS.dwg



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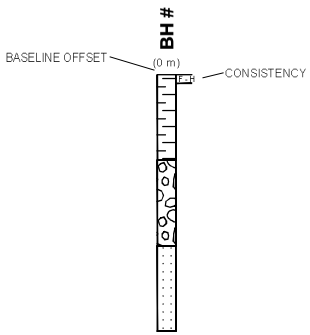

MATERIAL GRAPHIC

SAND

LEGEND

- DUNE SAND
- ESTUARINE DEPOSIT
- LIMESTONE
- INFERRED BOUNDARY
- NATURAL SURFACE

Geological boundaries are only known at the test site locations and have been inferred between the test sites. These geological boundaries have been provided to assist with the geological interpretation and should not be considered to represent actual boundaries that may vary from these lines.

H 1:1000

0 10 20 30 40 50m

V 1:100

0 1 2 3 4 5m

drawn	AMG
approved	RM
date	25/09/2014
scale	1:1000H 1:100V
original size	A3

coffey

client: JDSi CONSULTING ENGINEERS

project: STAGE 3 - THE STABLES EAST PORT HEDLAND, WA

title: SUB-SURFACE SECTION C-C'

project no: GEOTPER10160AA | fig no: DRAWING 3 | rev:

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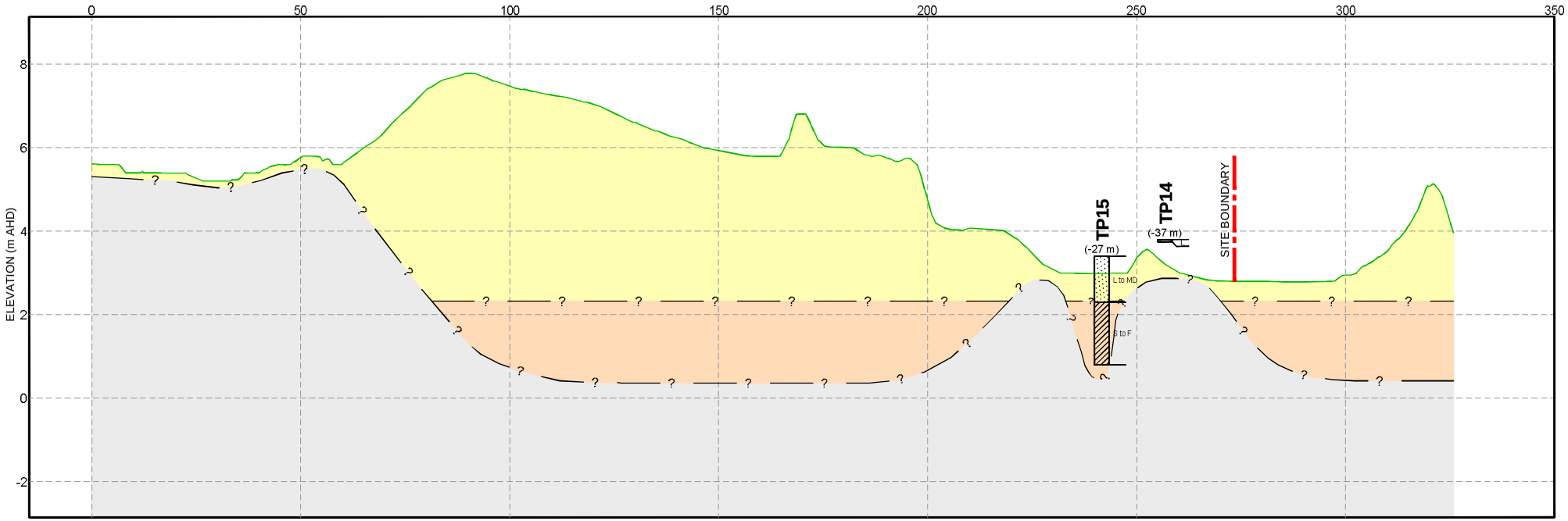

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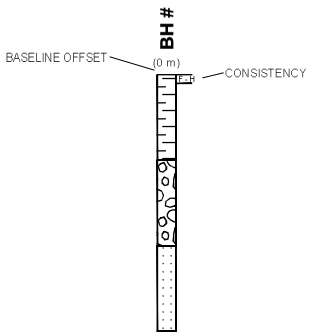

MATERIAL GRAPHIC

SAND

CLAY

LEGEND

DUNE SAND

ESTUARINE DEPOSIT

LIMESTONE

INFERRED BOUNDARY

NATURAL SURFACE

Geological boundaries are only known at the test site locations and have been inferred between the test sites. These geological boundaries have been provided to assist with the geological interpretation and should not be considered to represent actual boundaries that may vary from these lines.

H 1:1000 (0, 10, 20, 30, 40, 50m)

V 1:100 (0, 1, 2, 3, 4, 5m)

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA
date	25/09/2014		title:	SUB-SURFACE SECTION D-D'
scale	1:1000H 1:100V		project no: GEOTPER10160AA	fig no: DRAWING 4 / rev:
original size	A3			

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Appendix A -
Test Pit Excavation Logs and Photographs

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	***TP1***
		Sheet No.	1 of 1
		Project No.	***GEOTPERT10160AA***
Client	: ***JDSi Consulting Engineers***	Date excavated	***11/9/14***
Principal	: ***Landcorp***	Date completed	***11/9/14***
Project	: ***Geotechnical Investigation Styles Road***	Logged by :	***JC***
Location	: ***East Port Hedland, WA***	Checked by :	***RM***

Position : E: 670602.911, N: 7752903.902 (50 MGA94) Surface Elevation : 3.9m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 0.50m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description: SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	Not Observed			0.0			**CLAYEY SAND**, fine to medium, brown; clay, low to medium plasticity; trace of , roots	D	MD			
					3.5	0.5			Testpit terminated on Limestone EXCAVATION TP1 TERMINATED AT 0.25 m					
					3.0	1.0								
					2.5	1.5								
					2.0	2.0								
					1.5	2.5								
					1.0									

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description

Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

TEST PIT 1

SPOIL EXCAVATED FROM TP1

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**TEST PIT EXCAVATION PHOTOGRAPH - TP1**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **APPENDIX A**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	*TP2*
		Sheet No.	1 of 1
		Project No.	*GEOTPERT10160AA*
Client	: *JDSi Consulting Engineers*	Date excavated	*11/9/14*
Principal	: *Landcorp*	Date completed	*11/9/14*
Project	: *Geotechnical Investigation Styles Road*	Logged by :	*JC*
Location	: *East Port Hedland, WA*	Checked by :	*RM*

Position : E: 670569, N: 7752944 (50 MGA94) Surface Elevation : 2.9m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 0.50m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	Not Observed	ASS ASS	2.5	0.0			**CLAYEY SAND**, fine to coarse, brown; clay, medium to high plasticity	M	L			
					2.0	0.5			Testpit terminated on Limestone EXCAVATION TP2 TERMINATED AT 0.40 m					
					1.5	1.0								
					1.0	1.5								
					0.5	2.0								
					0.0	2.5								

method
- N Natural Exposure
- X Existing Excavation
- BH Backhoe Bucket
- B Bulldozer Blade
- R Ripper
- E Excavator

support
- T Timbering

penetration
1 2 3 4 — minor resitance ranging to refusal

water
- 10 Oct., 73 Water Level on Date shown
- water inflow
- water outflow

samples & field tests
- U50 - Undisturbed Sample 50mm diameter
- U63 - Undisturbed Sample 63mm diameter
- D - Disturbed Sample
- B - Bulk Disturbed Sample
- E - Environmental Sample
- MC - Moisture Content
- HP - Hand Penetrometer (UCS kPa)
- VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
- PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture
- D - Dry
- M - Moist
- W - Wet
- W_P - Plastic Limit
- W_L - Liquid Limit

consistency / relative density
- VS - Very Soft
- S - Soft
- F - Firm
- St - Stiff
- VSt - Very Stiff
- H - Hard
- VL - Very Loose
- L - Loose
- MD - Medium Dense
- D - Dense
- VD - Very Dense

TEST PIT 2

SPOIL EXCAVATED FROM TP2

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**TEST PIT EXCAVATION PHOTOGRAPH - TP2**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **APPENDIX A**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	***TP3***
		Sheet No.	1 of 1
		Project No.	***GEOTPERT10160AA***
Client	: ***JDSi Consulting Engineers***	Date excavated	***11/9/14***
Principal	: ***Landcorp***	Date completed	***11/9/14***
Project	: ***Geotechnical Investigation Styles Road***	Logged by :	***JC***
Location	: ***East Port Hedland, WA***	Checked by :	***RM***

Position : E: 670450.02, N: 7752947.252 (50 MGA94) Surface Elevation : 3.2m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 0.60m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description: SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	Not Observed			0.0			**SANDY CLAY**, low to medium plasticity, brown; sand, fine to medimum grained; trace of , roots	D	F			
					3.0				Testpit terminated on Limestone EXCAVATION TP3 TERMINATED AT 0.05 m					
						0.5								
					2.5									
						1.0								
					2.0									
						1.5								
					1.5									
						2.0								
					1.0									
						2.5								
					0.5									

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description

Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

TEST PIT 3

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP3	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

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coffey geotechnics

Engineering Log - Excavation

		Excavation No.	*TP4*
		Sheet No.	1 of 1
		Project No.	*GEOTPERT10160AA*
Client	: *JDSi Consulting Engineers*	Date excavated	*11/9/14*
Principal	: *Landcorp*	Date completed	*11/9/14*
Project	: *Geotechnical Investigation Styles Road*	Logged by :	*JC*
Location	: *East Port Hedland, WA*	Checked by :	*RM*

Position : E: 670463, N: 7752984 (50 MGA94) Surface Elevation : 2.6m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.00m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	▽	ASS	2.5	0.0			**SANDY CLAY**, low to medium plasticity, grey brown and brown; sand, fine to medium grained	M	VS			
					2.0	0.5								
				ASS	1.5	1.0			Testpit terminated on Limestone EXCAVATION TP4 TERMINATED AT 1.00 m					
					1.0	1.5								
					0.5	2.0								
					0.0	2.5								

method
- N Natural Exposure
- X Existing Excavation
- BH Backhoe Bucket
- B Bulldozer Blade
- R Ripper
- E Excavator

support
- T Timbering

penetration
1 2 3 4 — minor resitance ranging to refusal

water
- 10 Oct., 73 Water Level on Date shown
- water inflow
- water outflow

samples & field tests
- U50 - Undisturbed Sample 50mm diameter
- U63 - Undisturbed Sample 63mm diameter
- D - Disturbed Sample
- B - Bulk Disturbed Sample
- E - Environmental Sample
- MC - Moisture Content
- HP - Hand Penetrometer (UCS kPa)
- VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
- PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture
- D - Dry
- M - Moist
- W - Wet
- W_P - Plastic Limit
- W_L - Liquid Limit

consistency / relative density
- VS - Very Soft
- S - Soft
- F - Firm
- St - Stiff
- VSt - Very Stiff
- H - Hard
- VL - Very Loose
- L - Loose
- MD - Medium Dense
- D - Dense
- VD - Very Dense

TEST PIT 4

SPOIL EXCAVATED FROM TP4

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP4	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

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coffey geotechnics

Engineering Log - Excavation

Excavation No.	TP5
Sheet No.	1 of 1
Project No.	GEOTPERT10160AA
Date excavated	11/9/14
Date completed	11/9/14
Logged by :	JC
Checked by :	RM

Client : **JDSi Consulting Engineers**

Principal : **Landcorp**

Project : **Geotechnical Investigation Styles Road**

Location : **East Port Hedland, WA**

Position : E: 670329, N: 7753030 (50 MGA94) Surface Elevation : 2.6m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.00m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetrometer kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N			2.5	0.0			**CLAY**, low to medium plasticity, brown; trace of , shell fragments throughout	M	VS			
					2.0	0.5								
					1.5	1.0			...becoming grey at 1.0m					
				ASS						W				
					1.0	1.5			Testpit terminated on Limestone EXCAVATION TP5 TERMINATED AT 1.50 m					
					0.5	2.0								
					0.0	2.5								

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

TEST PIT 5

SPOIL EXCAVATED FROM TP5

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**TEST PIT EXCAVATION PHOTOGRAPH - TP5**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **APPENDIX A**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

Excavation No.	TP6
Sheet No.	1 of 1
Project No.	GEOTPERT10160AA
Date excavated	11/9/14
Date completed	11/9/14
Logged by :	JC
Checked by :	RM

Client : **JDSi Consulting Engineers**

Principal : **Landcorp**

Project : **Geotechnical Investigation Styles Road**

Location : **East Port Hedland, WA**

Position : E: 670218, N: 7753068 (50 MGA94) Surface Elevation : 2.4m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.20m long 0.50m wide

excavation information | **material substance**

method	penetration	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetrometer kPa	Blows/ 150mm	structure and other observations
E	VE E F H VH	N		ASS	2.0	0.0			**CLAY**, low to medium plasticity, brown; trace of , shell fragments; trace of , roots (mangrove roots)	M	VS	100 200 300 400		
					1.5	0.5								
					1.0	1.0				W				
					0.5	1.5			Testpit terminated on Limestone EXCAVATION TP6 TERMINATED AT 1.50 m					
					0.0	2.0								
					-0.5	2.5								

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

TEST PIT 6

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP6	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	TP7
		Sheet No.	1 of 1
		Project No.	GEOTPERT10160AA
Client	: JDSi Consulting Engineers	Date excavated	11/9/14
Principal	: Landcorp	Date completed	11/9/14
Project	: Geotechnical Investigation Styles Road	Logged by :	JC
Location	: East Port Hedland, WA	Checked by :	RM

Position : E: 670222, N: 7753014 (50 MGA94) Surface Elevation : 3.2m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.00m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	Not Observed	HP =200kPa HP =250kPa	3.0	0.0			**SANDY CLAY**, medium to high plasticity, brown; sand, fine to medium grained	M	H			
				BULK	2.5 2.0 1.5 1.0 0.5	0.5 1.0 1.5 2.0 2.5			Testpit terminated on Limestone EXCAVATION TP7 TERMINATED AT 0.28 m					

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

method
- N Natural Exposure
- X Existing Excavation
- BH Backhoe Bucket
- B Bulldozer Blade
- R Ripper
- E Excavator

support
- T Timbering

penetration
1 2 3 4 — minor resitance ranging to refusal

water
- 10 Oct., 73 Water Level on Date shown
- water inflow
- water outflow

samples & field tests
- U50 - Undisturbed Sample 50mm diameter
- U63 - Undisturbed Sample 63mm diameter
- D - Disturbed Sample
- B - Bulk Disturbed Sample
- E - Environmental Sample
- MC - Moisture Content
- HP - Hand Penetrometer (UCS kPa)
- VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
- PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture
- D - Dry
- M - Moist
- W - Wet
- W_P - Plastic Limit
- W_L - Liquid Limit

consistency / relative density
- VS - Very Soft
- S - Soft
- F - Firm
- St - Stiff
- VSt - Very Stiff
- H - Hard
- VL - Very Loose
- L - Loose
- MD - Medium Dense
- D - Dense
- VD - Very Dense

TEST PIT 7

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP7	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	***TP8***
		Sheet No.	1 of 1
		Project No.	***GEOTPERT10160AA***
Client	: ***JDSi Consulting Engineers***	Date excavated	***11/9/14***
Principal	: ***Landcorp***	Date completed	***11/9/14***
Project	: ***Geotechnical Investigation Styles Road***	Logged by :	***JC***
Location	: ***East Port Hedland, WA***	Checked by :	***RM***

Position : E: 670183.956, N: 7753019.448 (50 MGA94) Surface Elevation : 3.1m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.00m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	Not Observed	HP =250kPa HP =300kPa	3.0	0.0			**CLAY**, medium to high plasticity, brown	M	H			
					2.5	0.5			Testpit terminated on Limestone EXCAVATION TP8 TERMINATED AT 0.50 m					
					2.0	1.0								
					1.5	1.5								
					1.0	2.0								
					0.5	2.5								

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description

Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

TEST PIT 8

SPOIL EXCAVATED FROM TP8

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP8	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

Excavation No.	TP9
Sheet No.	1 of 1
Project No.	GEOTPERT10160AA
Date excavated	11/9/14
Date completed	11/9/14
Logged by :	JC
Checked by :	RM

Client : **JDSi Consulting Engineers**

Principal : **Landcorp**

Project : **Geotechnical Investigation Styles Road**

Location : **East Port Hedland, WA**

Position : E: 670046.922, N: 7753072.883 (50 MGA94) Surface Elevation : 3m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.50m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetrometer kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	▽		3.0	0.0			**SANDY CLAY**, low to medium plasticity, brown; sand, fine to medium grained	M	VS			
					2.5	0.5								
					2.0	1.0								
					1.5	1.5			Testpit terminated on Limestone EXCAVATION TP9 TERMINATED AT 1.10 m					
					1.0	2.0								
					0.5	2.5								

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

TEST PIT 9

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP9	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

Excavation No.	***TP10***
Sheet No.	1 of 1
Project No.	***GEOTPERT10160AA***
Date excavated	***11/9/14***
Date completed	***11/9/14***
Logged by :	***JC***
Checked by :	***RM***

Client : ***JDSi Consulting Engineers***

Principal : ***Landcorp***

Project : ***Geotechnical Investigation Styles Road***

Location : ***East Port Hedland, WA***

Position : E: 669953, N: 7753102 (50 MGA94) Surface Elevation : 3m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.50m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetrometer kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	▽		3.0	0.0			**SANDY CLAY**, low to medium plasticity, brown; sand, fine to medium grained	M	S			
					2.5	0.5								
					2.0	1.0								
				BLK	1.5	1.5			Testpit terminated on Limestone EXCAVATION TP10 TERMINATED AT 1.20 m					
					1.0	2.0								
					0.5	2.5								

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

TEST PIT 10

SPOIL EXCAVATED FROM TP10

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**TEST PIT EXCAVATION PHOTOGRAPH - TP10**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **APPENDIX A**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

Excavation No.	***TP11***
Sheet No.	1 of 1
Project No.	***GEOTPERT10160AA***
Date excavated	***11/9/14***
Date completed	***11/9/14***
Logged by :	***JC***
Checked by :	***RM***

Client : ***JDSi Consulting Engineers***

Principal : ***Landcorp***

Project : ***Geotechnical Investigation Styles Road***

Location : ***East Port Hedland, WA***

Position : E: 670151.558, N: 7753028.043 (50 MGA94) Surface Elevation : 3.1m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.50m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N	▽	HP =150kPa	3.0 / 2.5	0.0 / 0.5			**SANDY CLAY**, medium to high plasticity, brown; sand, fine to medium grained	M	VSt			
					2.0 / 1.5 / 1.0 / 0.5	1.0 / 1.5 / 2.0 / 2.5			Testpit terminated on Limestone EXCAVATION TP11 TERMINATED AT 0.70 m					

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

TEST PIT 11

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP11	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	***TP12***
		Sheet No.	1 of 1
		Project No.	***GEOTPERT10160AA***
Client	: ***JDSi Consulting Engineers***	Date excavated	***11/9/14***
Principal	: ***Landcorp***	Date completed	***11/9/14***
Project	: ***Geotechnical Investigation Styles Road***	Logged by :	***JC***
Location	: ***East Port Hedland, WA***	Checked by :	***RM***

Position : E: 670328.015, N: 7752993.545 (50 MGA94) Surface Elevation : 3.7m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 1.00m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
B		N	Not Observed	HP =350kPa		0.0			**CLAY**, medium to high plasticity, brown	D	H			
					3.5				Testpit terminated on Limestone EXCAVATION TP12 TERMINATED AT 0.10 m					
						0.5								
					3.0									
						1.0								
					2.5									
						1.5								
					2.0									
						2.0								
					1.5									
						2.5								
					1.0									

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

method
- N Natural Exposure
- X Existing Excavation
- BH Backhoe Bucket
- B Bulldozer Blade
- R Ripper
- E Excavator

support
- T Timbering

penetration
1 2 3 4 — minor resitance ranging to refusal

water
- 10 Oct., 73 Water Level on Date shown
- water inflow
- water outflow

samples & field tests
- U50 - Undisturbed Sample 50mm diameter
- U63 - Undisturbed Sample 63mm diameter
- D - Disturbed Sample
- B - Bulk Disturbed Sample
- E - Environmental Sample
- MC - Moisture Content
- HP - Hand Penetrometer (UCS kPa)
- VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
- PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture
- D - Dry
- M - Moist
- W - Wet
- W_P - Plastic Limit
- W_L - Liquid Limit

consistency / relative density
- VS - Very Soft
- S - Soft
- F - Firm
- St - Stiff
- VSt - Very Stiff
- H - Hard
- VL - Very Loose
- L - Loose
- MD - Medium Dense
- D - Dense
- VD - Very Dense

TEST PIT 12

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP12	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

Excavation No.	*TP13*
Sheet No.	1 of 1
Project No.	*GEOTPERT10160AA*
Date excavated	*11/9/14*
Date completed	*11/9/14*
Logged by :	*JC*
Checked by :	*RM*

Client : ***JDSi Consulting Engineers***

Principal : ***Landcorp***

Project : ***Geotechnical Investigation Styles Road***

Location : ***East Port Hedland, WA***

Position : E: 670960.932, N: 7752994.273 (50 MGA94) Surface Elevation : 6.2m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 2.00m long 0.50m wide

excavation information | **material substance**

method	penetration (VE, E, F, H, VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa (100, 200, 300, 400)	Blows/ 150mm	structure and other observations
E		N	Not Observed		6.0, 5.5, 5.0, 4.5, 4.0, 3.5	0.0, 0.5, 1.0, 1.5, 2.0, 2.5			**SAND**, fine to coarse, pale brown; trace of , silt; trace of , shell and shell fragments; trace of roots to 2.0m	D	L to MD			
									EXCAVATION TP13 TERMINATED AT 2.50 m					

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

TEST PIT 13

SPOIL EXCAVATED FROM TP13

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP13	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

Client	: *JDSi Consulting Engineers*	Excavation No.	*TP14*
Principal	: *Landcorp*	Sheet No.	1 of 1
Project	: *Geotechnical Investigation Styles Road*	Project No.	*GEOTPERT10160AA*
Location	: *East Port Hedland, WA*	Date excavated	*11/9/14*
		Date completed	*11/9/14*
		Logged by :	*JC*
		Checked by :	*RM*

Position : E: 671154.984, N: 7752923.005 (50 MGA94) Surface Elevation : 3.8m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 0.50m long 0.50m wide

excavation information | **material substance**

method	penetration (VE, E, F, H, VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetrometer kPa (100, 200, 300, 400)	Blows/ 150mm	structure and other observations
E		N	Not Observed			0.0			**SAND**, fine to coarse, pale brown; with some , fine to medium sub rounded to sub angular gravels	D	L			
									Testpit terminated on Limestone EXCAVATION TP14 TERMINATED AT 0.05 m					

RL (m): 3.5, 3.0, 2.5, 2.0, 1.5, 1.0

depth (m): 0.5, 1.0, 1.5, 2.0, 2.5

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

TEST PIT 14

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP14	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	TP15
		Sheet No.	1 of 1
		Project No.	GEOTPERT10160AA
Client	: JDSi Consulting Engineers	Date excavated	11/9/14
Principal	: Landcorp	Date completed	11/9/14
Project	: Geotechnical Investigation Styles Road	Logged by :	JC
Location	: East Port Hedland, WA	Checked by :	RM

Position : E: 671137, N: 7752922 (50 MGA94) Surface Elevation : 3.4m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 2.00m long 0.50m wide

excavation information | **material substance**

method	penetration	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetro-meter kPa	Blows/ 150mm	structure and other observations
E	VE E F H VH	N				0.0			**SAND**, fine to coarse, pale brown; trace of , sub rounded fine to medium gravels; trace of , shell and shell fragments	D	L to MD			
					3.0	0.5				M				
					2.5	1.0								
					2.0	1.5			**SANDY CLAY**, low plasticity, grey and green grey; sand, fine frained		S to F			
					1.5	2.0				W				
					1.0	2.5								
					0.5				EXCAVATION TP15 TERMINATED AT 2.60 m					

hand penetrometer scale: 100, 200, 300, 400

method

N Natural Exposure
X Existing Excavation
BH Backhoe Bucket
B Bulldozer Blade
R Ripper
E Excavator

support

T Timbering

penetration

1 2 3 4 — minor resitance ranging to refusal

water

10 Oct., 73 Water Level on Date shown
water inflow
water outflow

samples & field tests

U50 - Undisturbed Sample 50mm diameter
U63 - Undisturbed Sample 63mm diameter
D - Disturbed Sample
B - Bulk Disturbed Sample
E - Environmental Sample
MC - Moisture Content
HP - Hand Penetrometer (UCS kPa)
VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture

D - Dry
M - Moist
W - Wet
W_P - Plastic Limit
W_L - Liquid Limit

consistency / relative density

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

GEOTPERT_01.GLB Log EXCAVATION + PSP/DCP GEOTPERT10160AA.GPJ <<DrawingFile>> 30/09/2014 09:41

TEST PIT 15

TEST PIT 15

drawn	AMG	coffey	client:	JDSi CONSULTING ENGINEERS	
approved	RM		project:	STAGE 3 - THE STABLES EAST PORT HEDLAND, WA	
date	25/09/2014				
scale	NOT TO SCALE		title:	TEST PIT EXCAVATION PHOTOGRAPH - TP15 (PAGE 1 OF 2)	
original size	A4		project no: GEOTPER10160AA	fig no: APPENDIX A	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

SPOIL EXCAVATED FROM TP15

SPOIL EXCAVATED FROM TP15

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**TEST PIT EXCAVATION PHOTOGRAPH - TP15 (PAGE 2 OF 2)**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **APPENDIX A**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

coffey geotechnics

Engineering Log - Excavation

		Excavation No.	***TP16***
		Sheet No.	1 of 1
		Project No.	***GEOTPERT10160AA***
Client	: ***JDSi Consulting Engineers***	Date excavated	***11/9/14***
Principal	: ***Landcorp***	Date completed	***11/9/14***
Project	: ***Geotechnical Investigation Styles Road***	Logged by :	***JC***
Location	: ***East Port Hedland, WA***	Checked by :	***RM***

Position : E: 670703.096, N: 7753137.713 (50 MGA94) Surface Elevation : 2.8m (AHD)

Equipment type : 5t Excavator Method : Bucket Excavator Excavation dimensions : 2.00m long 0.50m wide

excavation information | **material substance**

method	penetration (VE E F H VH)	support	ground water	samples & field tests	RL (m)	depth (m)	graphic log	classification symbol	material description (SOIL TYPE, Plasticity or Particle Characteristic, Colour, Secondary and Minor Components)	moisture condition	consistency / relative density	hand penetrometer kPa (100 200 300 400)	Blows/ 150mm	structure and other observations
E		N				0.0			**CLAY**, low to medium plasticity, brown	M	S to F			
					2.5									
						0.5			**SANDY CLAY**, low to medium plasticity, grey; sand, fine to medium grained					
					2.0									
						1.0								
					1.5									
						1.5								
					1.0									
						2.0			EXCAVATION TP16 TERMINATED AT 2.00 m	W				
					0.5									
						2.5								
					0.0									

method
- N Natural Exposure
- X Existing Excavation
- BH Backhoe Bucket
- B Bulldozer Blade
- R Ripper
- E Excavator

support
- T Timbering

penetration
1 2 3 4 — minor resitance ranging to refusal

water
- 10 Oct., 73 Water Level on Date shown
- water inflow
- water outflow

samples & field tests
- U50 - Undisturbed Sample 50mm diameter
- U63 - Undisturbed Sample 63mm diameter
- D - Disturbed Sample
- B - Bulk Disturbed Sample
- E - Environmental Sample
- MC - Moisture Content
- HP - Hand Penetrometer (UCS kPa)
- VS - Vane Shear; P-Peak, R-Remouded (uncorrected kPa)
- PBT - Plate Bearing Test

classification symbols & soil description
Based on Unified Classification System

moisture
- D - Dry
- M - Moist
- W - Wet
- W_P - Plastic Limit
- W_L - Liquid Limit

consistency / relative density

VS	- Very Soft
S	- Soft
F	- Firm
St	- Stiff
VSt	- Very Stiff
H	- Hard
VL	- Very Loose
L	- Loose
MD	- Medium Dense
D	- Dense
VD	- Very Dense

TEST PIT 16

SPOIL EXCAVATED FROM TP16

drawn	**AMG**	coffey	client:	**JDSi CONSULTING ENGINEERS**	
approved	**RM**		project:	**STAGE 3 - THE STABLES EAST PORT HEDLAND, WA**	
date	**25/09/2014**				
scale	**NOT TO SCALE**		title:	**TEST PIT EXCAVATION PHOTOGRAPH - TP16**	
original size	**A4**		project no: **GEOTPER10160AA**	fig no: **APPENDIX A**	rev:

DWG: G:\GEOT\GEOTPERT\10000\10160AA Stage 3, East Port Hedland\7. Drawings\GEOTPERT10160AA - TEST PIT PHOTOS - APPENDIX A.dwg

Appendix B -
Laboratory Test Results

Welshpool, Perth Laboratory

Coffey Testing Pty Ltd
ABN 92 114 364 046
269A Treasure Road (Cnr Poole St)
Welshpool WA 6106

Phone: +61 8 6466 2400
Fax: +61 8 6466 2450

Report No: WELS14S-06336-1
Issue No: 1

Material Test Report

Client:	Coffey Geotechnics Pty Ltd (Burswood) 53 Burswood Road Burswood WA 6100
Principal:	JDSi
Project No.:	INFOWELS01719AA
Project Name:	GEOTPERT10160AA - Stage 3, East Port Hedland
Lot No.: NA	**TRN:** NA

Accredited for compliance with ISO/IEC 17025.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.

Approved Signatory: Jonathan Barry (Geotechnician)
NATA Accredited Laboratory Number:431
Date of Issue: 24/09/2014

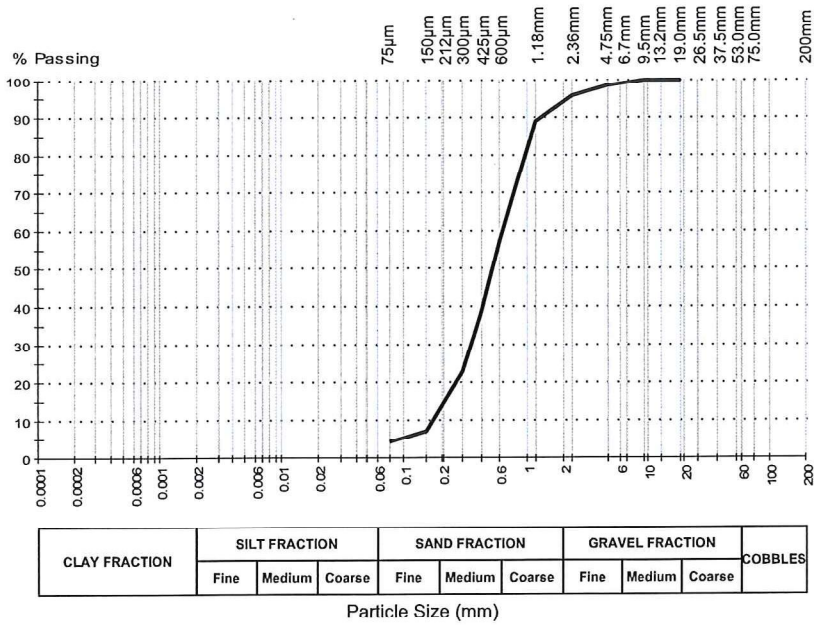
Sample Details

Sample ID:	WELS14S-06336
Client Sample:	permeability 1
Date Sampled:	
Source:	Unknown
Material:	
Specification:	Determined by client
Sampling Method:	Submitted by client
Project Location:	Styles Road, Port Headland
Sample Location:	permeability 1

Other Test Results

Description	Method	Result	Limits

Particle Size Distribution



Method: AS 1289.3.6.1
Drying by: Oven
Date Tested: 22/09/2014
Note:

Sieve Size	% Passing	Limits
19.0mm	100	
9.5mm	100	
4.75mm	99	
2.36mm	96	
1.18mm	89	
600µm	57	
425µm	39	
300µm	23	
150µm	7	
75µm	4	

Comments

Sample supplied by client.

Welshpool, Perth Laboratory

Coffey Testing Pty Ltd
ABN 92 114 364 046
269A Treasure Road (Cnr Poole St)
Welshpool WA 6106

Phone: +61 8 6466 2400
Fax: +61 8 6466 2450

Report No: WELS14S-06337-1
Issue No: 1

Material Test Report

Client: Coffey Geotechnics Pty Ltd (Burswood)
53 Burswood Road
Burswood WA 6100

Principal: JDSi
Project No.: INFOWELS01719AA
Project Name: GEOTPERT10160AA - Stage 3, East Port Hedland
Lot No.: NA **TRN:** NA

Accredited for compliance with ISO/IEC 17025.

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.

NATA
WORLD RECOGNISED ACCREDITATION

J. Barry

Approved Signatory: Jonathan Barry (Geotechnician)
NATA Accredited Laboratory Number:431
Date of Issue: 24/09/2014

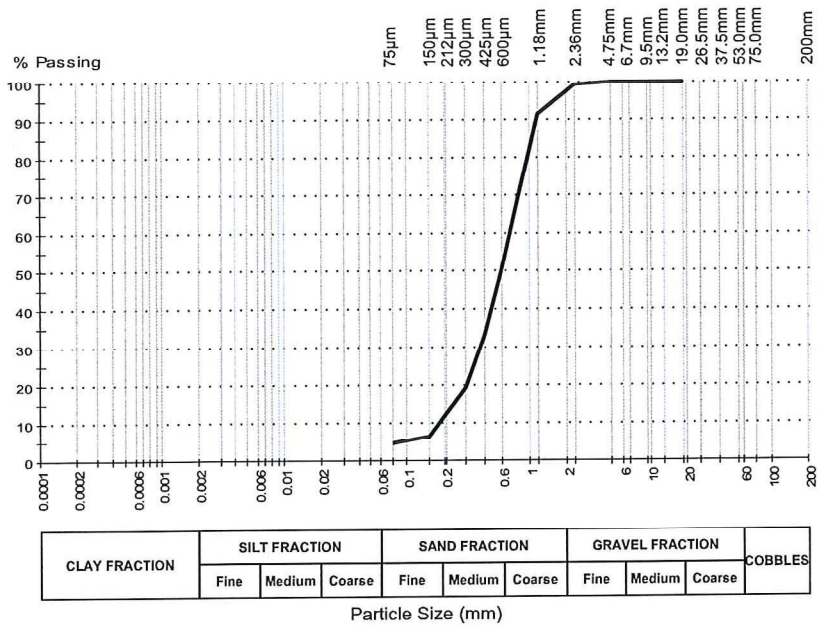
Sample Details

Sample ID:	WELS14S-06337
Client Sample:	Permeability 2
Date Sampled:	
Source:	Unknown
Material:	
Specification:	Determined by client
Sampling Method:	Submitted by client
Project Location:	Styles Road, Port Headland
Sample Location:	Permeability 2

Other Test Results

Description	Method	Result	Limits

Particle Size Distribution



Method: AS 1289.3.6.1
Drying by: Oven
Date Tested: 22/09/2014
Note:

Sieve Size	% Passing	Limits
19.0mm	100	
9.5mm	100	
4.75mm	100	
2.36mm	99	
1.18mm	91	
600µm	50	
425µm	33	
300µm	19	
150µm	7	
75µm	5	

Comments

Sample supplied by client.

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Stage 3 (The Stables)
East Port Hedland

APPENDIX C
INDICATIVE DESIGN CONCEPTS

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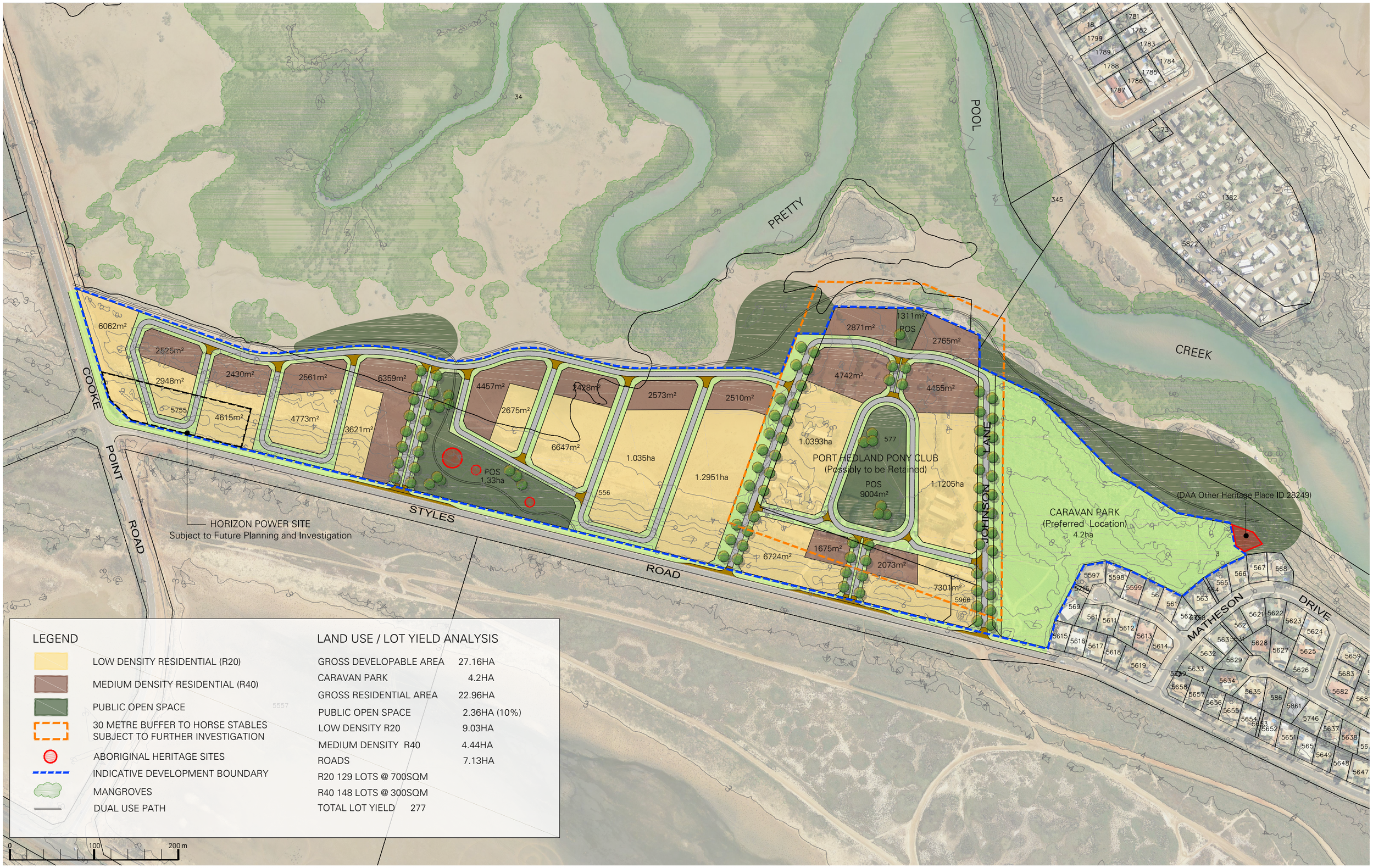


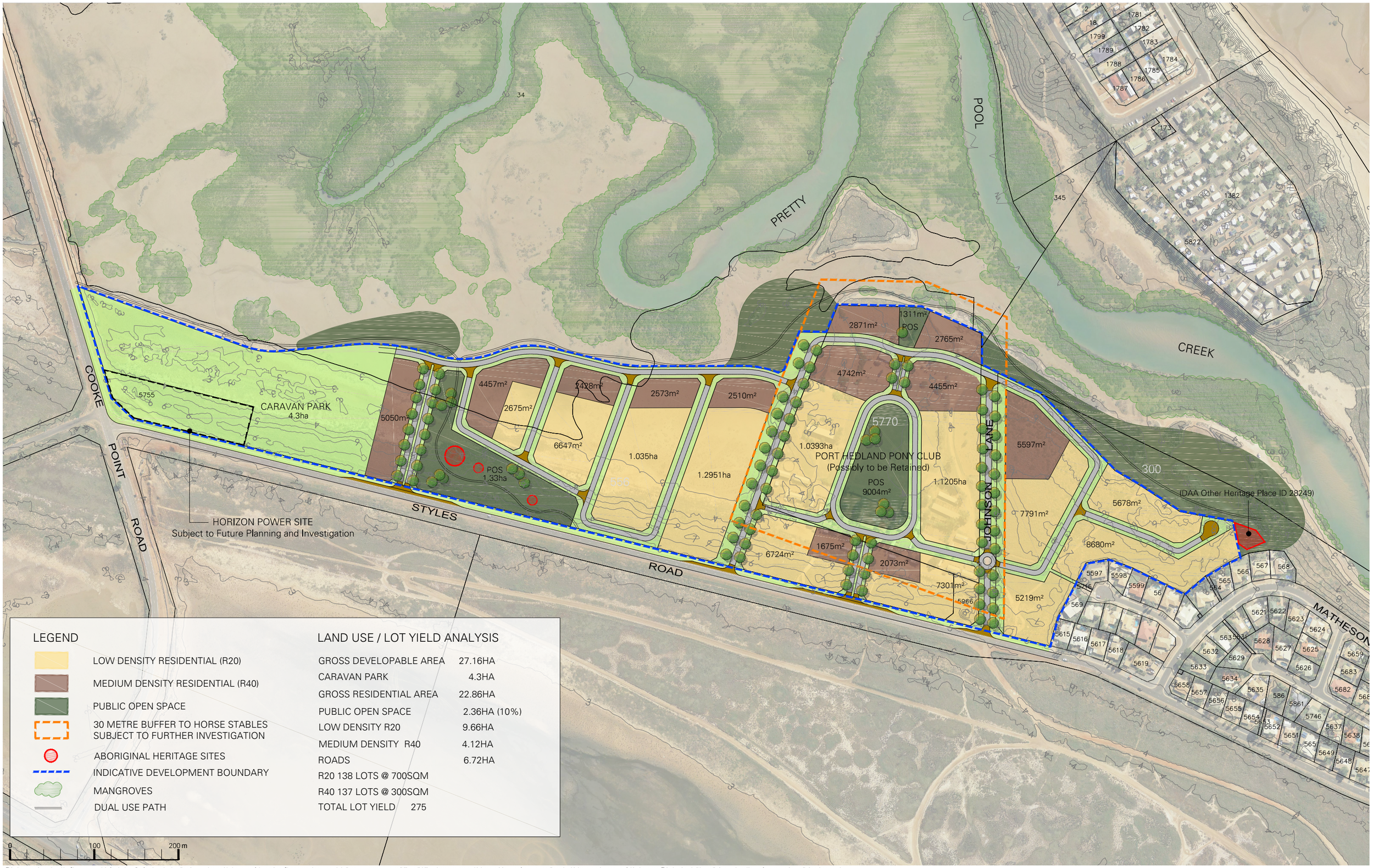
INDICATIVE DEVELOPMENT CONCEPT - OPTION 1

LOTS 556, 5770 AND 300 STYLES ROAD
PORT HEDLAND

NORTH

Date: 17.2.2015
Scale @ A3: 1:4000
Project / Plan
P006 CON 003A

RFF
CHESTERMAN



INDICATIVE DEVELOPMENT CONCEPT - OPTION 2

LOTS 556, 5770 AND 300 STYLES ROAD

PORT HEDLAND

NORTH

Date: 17.2.2015
Scale @ A3: 1:4000
Project / Plan
P006 CON 004A

RFF
CHESTERMAN

Stage 3 (The Stables)
East Port Hedland

APPENDIX D
WATER BALANCE

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Modelling Methodology

Water Balance

The water balance results given in Section 5.3.4 were calculated with the assumptions detailed below by conceptual development option.

Conceptual Development Option 1

1. Residential

Household type	No. of Lots	Households per lot	Average Area per lot (m²)	Irrigated area per lot (m²)
R20	199	1	500	120
R40	202	1	250	40
Cottage				
Apartment R20				
Apertment R40				

Percentage of lots with rainwater tanks (%)	0
Percentage of lots with greywater recycling systems (%)	0
Total residential area (m²)	150,000
Total irrigated area (m²)	31,960

4. Public Open Space (POS)

	Area (m²)	Amenities (kL/year)
Active POS	17,000	10
Other Irrigated POS		
Street scaping		
Non-irrigated POS	35,000	
Total POS area (m²)	52,000	
Total irrigated POS area (m²)	17,000	

Area Summary

Residential (m²)	150,000
Schools (m²)	0
Commercial and industrial (m²)	0
Public open space (m²)	52,000
Miscellaneous (m²)	0
Total development area (m²)	202,000

	DEMAND (kL/year)	REQUIRED SUPPLY (kL/year)	
		Drinking water	Non-drinking water
Residential indoor	52,713	43,224	9,488
Residential outdoor	19,408	2,135	17,273
Residential total	**72,121**	**45,359**	**26,762**
Public open space	**22,110**	**221**	**21,889**
Miscellaneous	**0**	**0**	**0**
Development total	**94,231 kL**	**45,580 kL**	**48,650 kL**
kL / person / year	**100.11**		

Conceptual Development Option 2

1. Residential

Household type	No. of Lots	Households per lot	Average Area per lot (m²)	Irrigated area per lot (m²)
R20	207	1	500	165
R40	187	1	220	65
Cottage				
Apartment R20				
Apertment R40				

Percentage of lots with rainwater tanks (%)	0
Percentage of lots with greywater recycling systems (%)	0
Total residential area (m²)	144,640
Total irrigated area (m²)	46,374

4. Public Open Space (POS)

	Area (m²)	Amenities (kL/year)
Active POS	18,000	10
Other Irrigated POS		
Street scaping		
Non-irrigated POS	35,000	
Total POS area (m²)	53,000	
Total irrigated POS area (m²)	18,000	

Area Summary

Residential (m²)	144,640
Schools (m²)	0
Commercial and industrial (m²)	0
Public open space (m²)	53,000
Miscellaneous (m²)	0
Total development area (m²)	197,640

	DEMAND (kL/year)	REQUIRED SUPPLY (kL/year)	
		Drinking water	Non-drinking water
Residential indoor	52,242	42,839	9,404
Residential outdoor	25,996	2,860	23,137
Residential total	**78,239**	**45,698**	**32,540**
Public open space	**23,410**	**234**	**23,176**
Development total	**101,649 kL**	**45,932 kL**	**55,716 kL**
kL / person / year	**108.96**		

Stage 3 (The Stables)
East Port Hedland

APPENDIX E
XPSWMM MODELLING

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Modelling Methodology

XPSWMM Modelling

Modelling was undertaken using XPSWMM, an industry standard hydrologic and hydraulic modelling software system. It routes flow from hydrological sub-catchments through 1D connection, allowing an analysis of the hydraulics of the Site drainage. All hydrological sub-catchments modelled use the Laurenson Routing Method, with a B value calculated by the XPSWMM software.

Rainfall Parameters

Design rainfall events for Port Hedland were determined following the procedure detailed in Australian Rainfall and Runoff *(*Engineers Australia 1997). Catchment areas and slopes are determined from analysis of topographical data. The catchment roughness and percentage imperviousness are conservative and were determined from a combination of field experiments, review of AR&R and other technical documents (e.g. Stream Channel Analysis (Water and Rivers Commission 2001)).

Pre-development Modelling

No modelling of the pre-development conditions was undertaken.

Post Development Modelling

The post development was modelled as 11 subcatchments for both indicative development concepts. Areas, slopes and runoff assumptions are provided in Table C1 and Table C2**Error! Reference source not found.**.

Topographic survey data covering the Site was provided by LandCorp through 0.5 m contours. This enabled the development of a Digital Elevation Model to define the existing overland flow paths. In some cases the overland flow paths have not been easily defined, engineering judgement has been applied to determine catchment sizes and expected flow paths. A 2 m grid cell size has been adopted. This was considered an appropriate resolution for the Site as it adequately represents topographical features such as roads and drains.

The aim of the post development modelling is to demonstrate the proposed drainage design is capable of detaining and infiltrating the first 15 mm of rainfall and to show that the development is able to safely convey the critical storm events through/around the Site.

The model nodes and links for the post development of conceptual development options 1 and 2 are shown in Figures A3 and A4, respectively. The points of interest in the model are the swale cells. The peak flow rate of water leaving the Site can be determined from these features. The post development model has been designed to detain the first 15mm of any storm event on lots and to convey the first 15 mm of any storm event on the road reserve to the swale system on the northern boundary of the Site for infiltration or, for rainfalls greater than 15 mm, overtop the swale.

The model nodes and links for both the conceptual option 1 and 2 models are shown in Figure C1 and Figure C2, respectively.

Delineation of contributing uses to lot areas for both R20 and R40 densities was based on the Western Australian Residential Design Codes (WAPC, 2013). Contributing uses for both R-densities are provided in Table C3.

Table C1 Post Development Catchment Breakup –Option 1

Sub-catchment	Total Area (ha)	Slope	Total Road Reserve	Road	Verge	Total Number of Lots	Developed Lot Areas (ha)			General Lot Areas (ha)		
							Roof	Paved	Garden	POS	Path	Caravan Park
A	2.28	0.001	0.58	0.29	0.29	40	0.89	0.22	0.43	-	0.05	-
B	3.20	0.001	1.15	0.57	0.57	68	1.37	0.24	0.46	-	-	-
C	0.68	0.001	-	-	-	0	-	-	-	0.59	0.05	-
D	5.13	0.001	1.33	0.67	0.67	66	1.40	0.30	0.59	1.27	0.11	-
E	4.39	0.001	1.41	0.71	0.71	69	1.61	0.47	0.90	-	-	-
F	0.95	0.001	-	-	-	0	-	-	-	0.90	0.04	-
G	0.98	0.001	-	-	-	26	0.32	0.08	0.16	0.13	0.04	-
H	7.30	0.001	2.58	1.29	1.29	122	2.30	0.74	1.43	0.89	-	-
I	4.20	0.001	-	-	-	0	-	-	-	-	-	4.20
J	2.56	0.001	-	-	-	0	-	-	-	2.47	0.81	-
K	2.53	0.015	0.31	0.15	0.15	0	0.90	-	1.35	-	-	-

Table C2 Post Development Catchment Breakup –Option 2

Sub-catchment	Total Area (ha)	Slope	Total Road Reserve	Road	Verge	Total Number of Lots	Developed Lot Areas (ha)			General Lot Areas (ha)		
							Roof	Paved	Garden	POS	Path	Caravan Park
A	4.62	0.001	-	-	-	0	-	-	-	-	0.06	4.40
B	1.11	0.001	0.24	0.12	0.12	18	0.45	0.14	0.28	-	-	-
C	0.63	0.001	-	-	-	0	-	-	-	0.59	0.04	-
D	6.08	0.001	1.77	0.88	0.88	90	1.82	0.33	0.65	1.27	0.11	-
E	4.40	0.001	1.41	0.71	0.71	69	1.61	0.47	0.91	-	-	-
F	0.96	0.001	-	-	-	0	-	-	-	0.90	0.04	-
G	0.98	0.001	-	-	-	26	0.48	0.04	0.07	0.13	0.04	-
H	7.53	0.001	2.58	1.29	1.29	113	2.43	0.55	1.07	0.89	-	-
I	3.04	0.001	0.67	0.34	0.34	63	1.25	0.41	0.79	-	-	-
J	2.56	0.001	-	-	-	0	-	-	-	2.49	0.08	-
K	2.54	0.015	0.31	0.15	0.15	45	0.90	-	1.35	-	-	-

Table C3 Lot Breakup by R-Density

R-Density	Average Lot Area (m^2)	Open Space (%)	Roof Area	Garden	Paved
R20	500	50%	250	66%	34%
R40	220	45%	121	66%	34%

Figure C1 XPSWMM Model Nodes and Links for Conceptual Development Option 1

Figure C2 XPSWMM Model Nodes and Links for Conceptual Development Option 2

About Cardno

Cardno is an ASX200 professional infrastructure and environmental services company, with expertise in the development and improvement of physical and social infrastructure for communities around the world. Cardno's team includes leading professionals who plan, design, manage and deliver sustainable projects and community programs. Cardno is an international company, listed on the Australian Securities Exchange [ASX: CDD].

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