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# Town of Port Hedland

## Report for South Hedland Flood Study

February 2011

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

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# 1. Introduction

## 1.1 Background

GHD Pty Ltd (GHD) was commissioned by the Town of Port Hedland (ToPH) to undertake a flood study to examine opportunities and constraints in regards to the drainage network in South Hedland and to develop a 5 year plan for upgrades and maintenance of drainage infrastructure. The study has been initiated in part to inform a Public Land Rationalisation Plan which is being developed to indentify potential development opportunities within the existing town area.

## 1.2 Study Objectives

The study objective is to provide the ToPH with information on the existing drainage systems in South Hedland and current levels of performance. Specific objectives of the study are to:

1. Create mapping and data for the existing drainage network and reservations.
2. Establish the functionality of the drainage network and recommend on improvements and/or areas of redundancy.
3. Recommend improvements to drainage network that maintain or improve the drainage function and enable more cost effective maintenance, whilst recognising the Town's desire to use these areas as linear open space in accordance with the POS guidelines.
4. Provide a methodology for determination of requests for access to drainage reserves with South Hedland and provide specific recommendations in regards to current requests before the Town.
5. Recommend priorities for upgrades or improvements.
6. Provide a 5 year overall drainage upgrade and maintenance plan, including estimated costs.
7. Identify potential funding sources for the upgrade or maintenance of the drainage network.

## 1.3 Climate Change

Climate change is an emerging issue, and although its existence is widely accepted, the effects are unclear and unquantifiable at the present time.

Potential climate change impacts may affect this study and some of these impacts could be significant. These could include, without limitation, impacts on the physical, climatic, commercial and/or social setting of the project. They may vary in magnitude, timing, duration and distribution and may have specific, cumulative and/or collateral impacts. These effects may impact on the operation, functionality, performance and durability of the study beyond what can be reasonably predicted with the current available knowledge. These issues relating to climate change have not been considered in this study. We have assumed that The Town of Port Hedland, in consultation with the State and Federal Governments, will address these issues separately to this flood study.

## 1.4 Sources of Information

Contours, aerial photography and cadastre were sourced from Landgate for the project. Survey of the drainage network was undertaken by AAM Surveys as a subcontractor to GHD to document the inverts and dimensions of existing structures, cross-sections of open drains and to identify any unique obstructions to the drainage network.

The town planning scheme and future land use planning documents were sourced from the Town of Port Hedland to inform the study.

# 2. Conceptual Review

## 2.1 General arrangement

South Hedland is generally flat with a gentle slope toward the north-west and north-east divided by a low ridge running north-south through the centre of the town. The ridge is an extension of naturally higher land south of the town and varying between 13 m and 16 m AHD. Throughout most parts of town lots drain overland to adjacent road reserves and roads are graded to direct stormwater to a network of open channels. Some recent developments have provided piped drainage systems to collect road runoff and discharge it to existing drains. The open channels convey stormwater through culverts and small bridges to the natural drainage line immediately west of the town site and to a large infiltration/evaporation drainage basin east of North Circular Road. Figure 2 illustrates the variation in elevation across the town and the network of open drains that currently exist.

In towns with relatively low relief such as South Hedland, a traditional piped drainage system does not present a practical solution. The low frequency of storms results in significant accumulation of mobile sediment within the catchment which will quickly blocks pipes and inlet structures. For other projects in the north west, the Department of Water has advised (discussed further Section 3.2) that in favourable environmental conditions, the ideal drainage network is characterised by the use of kerbed roads as the initial conveyor of stormwater, with kerb breaks located at topographic low points discharging stormwater to large open channels to safely convey stormwater away from the urban zone. The dominant nature of the drainage network in South Hedland is consistent with this strategy and provides significant advantages over a traditional piped drainage system. Design criteria for this preferred drainage strategy is explored in more detail in Section 3.

## 2.2 Existing Drainage System

On ground survey and site inspections were performed to document the nature of open drains and details of culverts and other structures that might have potential to impact on the hydraulic performance of the drainage network. The majority of observed structures are culverts and small bridges at road crossings and crossovers. Various other culverts and small bridges provide pedestrian crossings of the open drains.

The open drains are generally trapezoidal and vary in size and depth. Typically drains are between 1m and 2m deep and have a base width which varies between 2m and 4m although there are some exceptions. Longitudinal grades are very low with most open drains less than 0.5%. Figures 5a to 5f document the layout of drainage throughout the study area and the schedules in Appendix B document the dimensions of observed structures.

Site observations revealed that while some culverts are in good condition, a large number of culverts are obstructed by sediment and/or debris. Plate 1 illustrates the various degrees of obstruction which was typical for most culverts across the study area. These obstructions have the potential to severely limit the hydraulic capacity of the system resulting in flooding of the upstream drainage reserve, adjacent properties and flooding or overtopping of the road.


**Plate 1 – Culvert Obstructions (SH18, SH42, SH34 and SH57)**

Sedimentation occurs at places where water slows down and the velocity of flow is not sufficient to carry material that was made mobile by erosion upstream. This can occur at the entrance to culverts, where flow is restricted and the water slows down to get through the culvert. The general sedimentation of drains will also occur where flow from road gutters and small drains carries sediment to larger open drains where flow velocity is relatively low.

Obstruction by debris will occur more severely where culverts are small and and/or grates prevent the passage of smaller items, as observed at SH57 (Plate 1). Notwithstanding, a build up of debris can also occur at other culverts where larger objects such as tree branches or shopping trolleys catch smaller pieces of debris.

## 2.3 Proposed developments

The Port Hedland Public Land Rationalisation Plan (PLRP) provides an analysis of current and future growth and considers that release of urban land in Port Hedland and/or South Hedland is necessary to cater for future growth. The PLRP considers that:

> *"The growth in the Town will continue to be significant as BHP Billiton and other mining operations continue to expand and the Town of Port Hedland continues to invest in infrastructure and local facilities to support growth and decrease the proportion of fly-in fly-out*

*workers." … "population analysis suggest[s] potential growth of up to 40 percent within five years."*

The PLRP identifies parcels of public land throughout the study area which may be suitable for development.

A large number of the parcels identified for future development contain existing open drains or are adjacent to existing drainage reserves. Any development of land within drainage reserves must consider the potential impact on the drainage system and address any increases in flooding risk. The drainage reserves in South Hedland are critical in providing a safe passage for floodwaters to ensure the protection to property and infrastructure during large rainfall events. The function of the drainage reserves is akin to a "floodway" in that they provide flow paths for major events. Notwithstanding, alternative flow paths (floodways) may become relevant when the capacity of the drainage network is exceeded. Estimated peak flood levels throughout the study area give Council guidance to assess developments proposals and their engineering solutions to flood management.

## 2.4 Policy considerations and design criteria

The recommended floodplain management strategy considers that development within a floodway is considered obstructive to major flows and is not acceptable.

Notwithstanding, it could be considered that some types of development (roads, car parks and some park embellishments) within the drainage reserve maintain the "floodway", and then the Town of Port Hedland could set a criteria under which it will allow such developments encroach on the existing drainage reserves.

Developments within the floodway (such as car parks and footpaths) may encourage pedestrians to be within the major flow path during large rainfall events. Consequently, design of any development within or adjacent to drainage reserves will need to consider the safety of pedestrians.

## 2.5 Flooding from External Sources

This project has been undertaken in accordance with the guidelines given in Australian Rainfall and Runoff (Pilgrim 2001). It should be noted that there are no new publically available predictions that include climate change impacts. Sea level modelling and sensitivity assessment to climate change were not undertaken.

The Greater Port Hedland Storm-surge Study (GPHSS) was undertaken by Global Environmental Modelling Systems on behalf of the Ministry of Planning in 2000 and was used to inform the Port Hedland Area Planning Study (PHAPS); subsequently published by the Western Australian Planning Commission in 2003. The GPHSS provides flood mapping from the combination of runoff and storm surge in major rivers and creeks in the Port Hedland Area. In order to do so, statistical methods were used predict the peak flows major creeks and rivers in the area. The study predicted peak flow in South Creek during the 10-year and 100-year ARI as 90 $m^3/s$ and 383 $m^3/s$ respectively.

The GPHSS reports that peak storm surge occurs during a different type of storm (duration and intensity) than that which creates the peak catchment response and concludes that it is not appropriate to consider storm surge as a hydraulic constraint to flooding in streams and rivers. As such, the peak flow estimates for South Creek were considered to establish a water level downstream of the South Hedland drainage system.

## 2.6 Public Open Space

The Town of Port Hedland has Public Open Space Guidelines which provide Council and developers with guidance by way of a hierarchy for the distribution and embellishment of public open space (POS). In addition to the two District Open Spaces at Kevin Scott Reserve in South Hedland and Spoil Bank in Port Hedland; the guidelines identify a desired distribution of Local and Neighbourhood POS to provide opportunities within communities for informal recreation activities.

In South Hedland, drainage reserves and public access ways contribute significantly to open space provision providing opportunities for lineal recreation. As part of a program for urban renewal in South Hedland the ToPH along with the Department of Housing and South Hedland New Living have prepared “Development and Management Guidelines for Drainage Reserves and Public Access Ways” (the Guidelines). These guidelines propose a set of principles for redevelopment and management of existing drainage reserves to create viable public open spaces.

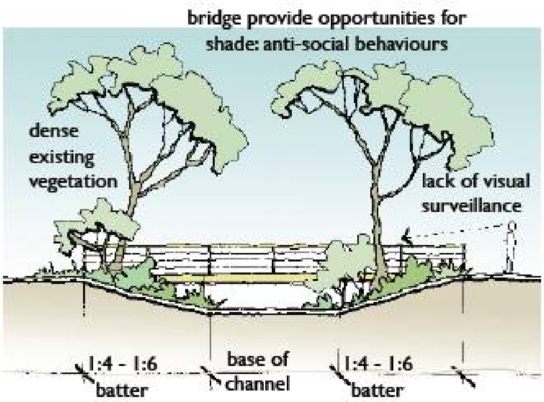
The principles advocate various improvements to allow better management of drainage reserves. Plate 2 and Plate 3 illustrate some typical treatments and opportunities considered by the Guidelines.

The Guidelines propose to improve the viability of drainage reserves as POS by interventions that will create three types of open space. They are:

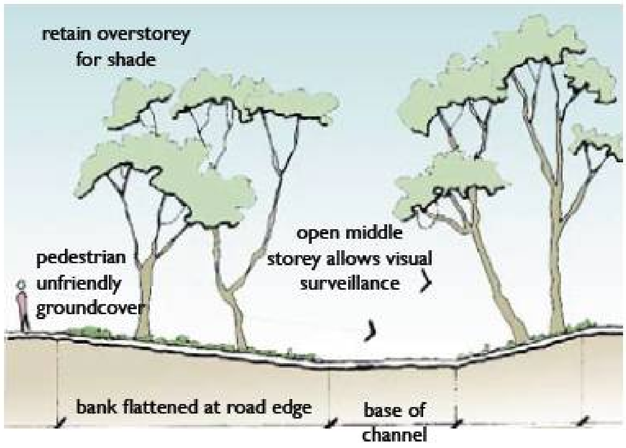
- Type 1 (local native parks), where drainage reserves are remediated to improve safety, security, maintenance and biodiversity;
- Type 2 (local connector parks), where as well as the treatments for Type 1 parks, redevelopment also includes provision of formal lineal recreation opportunities such as footpaths, lighting and minor embellishments (shade and seating etc); and
- Type 3 (local destination parks), where redevelopment will provide irrigated spaces and landscaped areas (eg: kick about, amphitheatre, public art).

The Public Open Space Guidelines would classify the suggested Type 3 embellishment as a “Neighbourhood Park” and suggest that they provide for a catchment of 1 km. Notwithstanding, it is understood that further analysis and consultation is currently being undertaken to identify the appropriate location for Type 2 and Type 3 redevelopments.

The hydraulic analysis undertaken as part of this study will help to identify constraints and opportunities to the proposed redevelopment of drainage reserves. More broadly, the extent of predicted flooding and flow velocities calculated as part of the flood study will allow informed consideration of proposals for development of the drainage reserves to improve their viability as POS. Further discussion as to the hydraulic impact of the proposed redevelopment options is included in Section 3.

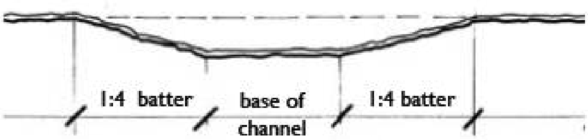



TYPICAL EXISTING CHANNEL SECTION

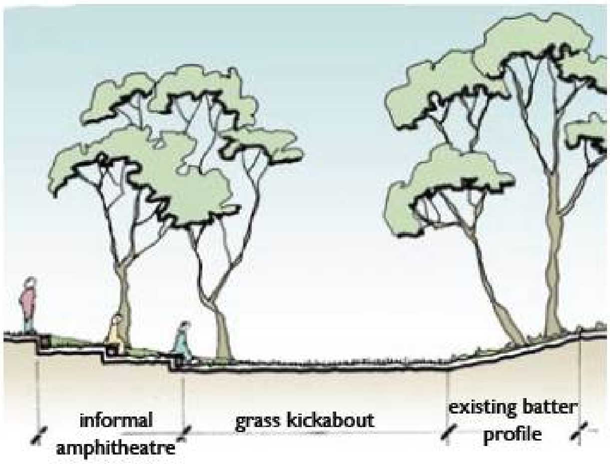



TYPICAL PROPOSED CHANNEL SECTION TO INCREASE VISUAL SURVEILLANCE TO 'HOT SPOTS'

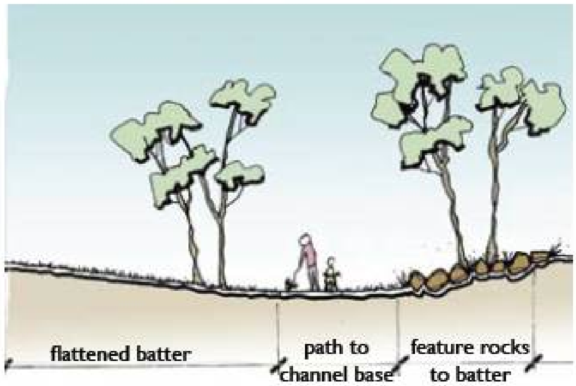
**Plate 2 – Proposed improvements for surveillance, (ToPH / MNLA, 2010)**




TYPICAL TRAPEZIODAL CHANNEL SECTION




TERRACING TO BATTER




ROCK LINING TO BATTER

**Plate 3 – Other potential opportunities, (ToPH, MNLA, 2010)**

# 3. Drainage Strategy

The north west of Western Australia experiences unreliable and highly variable rainfall. Precipitating mainly in the summer months, rainfall occurs as a result of the Northern Australian wet season and often occurs as a result of tropical cyclones. Consequently, much of the north west region is subject to major flooding during cyclonic events.

The treatment of stormwater to improve water quality in the North West region has been largely neglected, because removal of stormwater away from key infrastructure has been the main priority. Consequently, there is considerable scope available for the implementation of Water Sensitive Urban Design (WSUD) measures, to improve water quality in this part of the State.

In effort to incorporate the principles of WSUD, new development in the south of the state has included source infiltration and treatment of stormwater with runoff from constructed impervious areas retained or detained through the use of devices such as soak wells, pervious paving, vegetated swales, gardens or rainwater tanks. A modified approach to WSUD is required in the North West Region due to very different climatic conditions.

## 3.1 Water Sensitive Urban Design

Water Sensitive Urban Design (WSUD) can be defined as a design philosophy that provides a framework for water resource management in an urban context by integrating stormwater, wastewater and water supply. The key elements of water resource management in this context include protection from flooding, management of water quantity and quality to achieve ecological objectives and water conservation, efficiency and use.

The Western Australian Stormwater Management Manual lists nine objectives that relate to the management of stormwater. They are:

1. Water Quality - To maintain or improve the surface and groundwater quality within the development areas relative to pre development conditions.
2. Water Quantity - To maintain the total water cycle balance within development areas relative to the predevelopment conditions.
3. Water Conservation - To maximise the reuse of stormwater.
4. Ecosystem Health - To retain natural drainage systems and protect ecosystem health.
5. Economic Viability - To implement stormwater management systems that are economically viable in the long term.
6. Public Health - To minimise the public risk, including risk of injury or loss of life, to the community.
7. Protection of Property - To protect the built environment from flooding and water logging.
8. Social Values - To ensure that social, aesthetic and cultural values are recognised and maintained when managing stormwater.

9. Development - To ensure the delivery of best practice stormwater management through planning and development of high quality developed areas in accordance with sustainability and precautionary principles.

It is the intention of the DoW that WSUD principles should be applied to the whole of Western Australia. Modern urban water drainage design in Perth follows strategies set out in Better Urban Water Management (DoW 2008) which provides guidance to water management planning for the Swan Coastal Plain. Whilst the WSUD principles apply to South Hedland, the methods of detention and retention vary due to the climatic constraints.

The proposed principles outlined below incorporate appropriate WSUD and Best Management Practices (BMP) in the local context. In regards to water quantity (point 2 above), detention systems should be designed such that peak flows generated in the critical 1 yr ARI event should be preserved, whilst events greater than this can overflow off site via appropriate flow paths (DoW 2009).

## 3.2 Regional drainage principles

Drainage design in all areas of the State is engineered to the same design guidelines as outlined in *Local Government Guidelines for Subdivisional Development Edition 2 – 2009* (IPWEA 2009). However the type of drainage implemented varies. An overview of drainage implemented in north west regional centres is provided in Table 1 below, highlighting the variability in design.

Due to the large volumes of water generated in cyclonic events, the priority for stormwater management in the north west region of WA has been the rapid removal of stormwater away from infrastructure to avoid flood related damages. Some town centres have addressed this issue well by constructing a drainage network that is designed to rapidly remove stormwater. Others, such as Onslow, are constrained by site specific issues such as lack of relief town layout and have been forced to construct a drainage network with limited function.

In the north of Western Australia, given favourable environmental conditions, the ideal drainage network is represented by utilising kerbed roads as the initial conveyor of stormwater, with kerb breaks located at topographic low points discharging stormwater to large open channels that discharge stormwater away from the urban zone. Evidence suggests that treatment of stormwater in the North West region has not been required for the protection of downstream ecosystems. This is due mainly to the intensity of major event rainfall and the use of overland flow as the principle conveyance method. Existing Water Sensitive Urban Design measures for water quality improvement in this part of the State, relate to retaining or slowing frequent events in vegetated overland flow paths.

Evidence suggests that traditional piped drainage systems are not a practical solution to drainage management in the North West Region (as discussed previously under Heading 2.1). Where possible, development should be designed such that roads and open drains provide drainage via overland flow.

**Table 1 Adopted Drainage Principles in Other Centres**

| Design Rainfall Event | Karratha Gap Ridge Industrial Estate[1;5] | Broome North[2;5] | Newman Townsite[3] | Dampier Townsite[4] | Onslow |
|---|---|---|---|---|---|
| 1 yr ARI | Runoff collected in roadside swales | Runoff collected in open channel swale detention system | Pit and pipe system in townsite | Runoff contained within open stormwater drains / channels | Pit and pipe system in townsite |
| 5 yr ARI | Road runoff discharging to stormwater flood storage areas and open channels prior to discharge to coastal foreshore | Runoff contained within kerbs in road system | Runoff contained within kerbs in road system | Runoff contained within open stormwater drains / channels | Runoff conveyed in pits and pipes and roads to infiltration basins and marine outfalls |
| 100 yr ARI | Conveyed / Contained within road reserve / drainage reserve | Conveyed / Contained within road reserve | Discharged to large open channels | Conveyed / Contained within road / drainage reserve | Discharged to marine outfalls; Contained within infiltration basins |

1 GHD 2010; 2 GHD 2009; 3 JDSI 2010; 4 GHD 2010 5 DoW Approved

It is important to note that the South Hedland drainage network has proven to be an effective drainage system that performs its function well and that modification of the general arrangement and approach to drainage management is not recommended. Any proposed development that impinges on the drainage network is therefore not recommended, with the exception of additional landscaping, which may be suitable in some drainage reserves to enhance the town aesthetics, provided there is no major modification to the drainage infrastructure.

The IPWEA 2009 provides some guidance on drainage management. The drainage management guidelines contained therein provide a set of design criteria for water quantity and quality. The most applicable design criteria to the South Hedland town site will be a mix of urban and rural design criteria proposed by guidelines. Considering climatic constraints and the recommendations of IPWEA 2009, the following design criteria may be appropriate:

- The drainage system should be designed for the 100yr ARI rainfall event using overland flow.
- Arterial drains and compensating basins should be designed to contain the 5-year ARI rainfall event.
- Maximum flow velocities should not exceed 1 m/s in unlined open channels and 2 m/s in lined drains (can be arrested by the inclusion of drop structures);

- Mortared stone pitching shall be provided in open drains at all junctions and bends greater than 22.5$^{o}$; and
- Detention storage areas may be provided at suitable locations (can be on line) to reduce peak flow rates to the capacity of downstream facilities.
- Runoff from constructed impervious areas should be retained where possible within the lot or road reserve for the 1 yr ARI event;
- Provision shall be made using overland flow paths and storage facilities for peak 1 in 100 year storm event such that the floor level of all buildings shall be a minimum of 300 mm above the 100 year storm event.

In regards to the management of arterial drains the guidelines provide a "Recommended Floodplain Development Strategy" (Figure 4.3, pp 85 of IPWEA 2009) which is reproduced in this report as Figure 1. In regards to development within a floodway it recommends:

> *"Development (i.e., filling, building, etc) that is located within the floodway and is considered obstructive to major flows is not acceptable as it would increase flood levels upstream. No new dwellings are acceptable within a floodway."*

Figure 1 suggests that the increase in 100 yr flood level as a result of flood fringe development should be no more than 0.15 m and that habitable floor levels should be 0.5 m above that level.

To maintain the safety of pedestrians during flood events, Australian Rainfall and Runoff (Pilgram 2001) suggests that:

> "*the product of velocities and depths in streets and major flow paths should not exceed 0.4* $m^2/s$*"*.

This safety criterion should be applied to flow in streets and other areas where infrastructure may encourage pedestrians to be within the major flow path during large rainfall events.

Information for this section was developed using the following references:

- IPWEA 2009, Local Government Guidelines for Subdivisional Development, Edition 2
- GHD 2010. Report for Shire of Ashburton, Onslow Drainage Assessment (draft).
- GHD 2010. Report for Gap Ridge Industrial Estate, Urban Water Management Plan, April 2010.
- GHD 2009. Report for Broome North, Local Water Management Strategy, October 2009.
- JDSI 2010. Personal communications with JDSI.
- GHD 2010. Dampier Drainage Review (draft).

# 4. Development of Drainage Reserves

## 4.1 Key Considerations

As discussed in Section 2.4 and 3.2, any development within or adjacent to drainage reserves should be considered in regards to potential impact on infrastructure, receiving environments and public safety.

Key considerations are:

- Impact of the development on potential flood levels and resulting risk of damage to property and infrastructure.
- Predicted flow velocity resulting in changes to scour potential.
- Risk to public safety by changing access arrangements; encouraging pedestrians and/or vehicles to be in the floodway during rainfall events.
- Opportunities to incorporate water quality treatment measures that can provide protection to receiving environments.

It is recommended that all proposals for development within or adjacent to drainage reserves should be required to address these considerations.

Hydraulic models of varying complexity can be used to calculate the impact of proposals and estimate velocities of flow in different parts of the cross section / proposal. It is recommended that Council require a hydraulic assessment by a suitably qualified engineer for all development proposals within the floodway. Peak flow rates and water levels presented in Schedule 2 and Schedule 3 (Appendix B) can be used as inputs to such analysis.

The discussion below, presents a conceptual review of some key interventions that might be considered as part of the urban renewal program.

## 4.2 Changes to drain cross section

In simple terms the discharge capacity of a drain is proportional to the cross sectional area of the flow path and the difference between upstream and downstream water levels.

Therefore for a set discharge rate, a decrease in the cross section of flow can result in an increase in potential flood depth upstream. Further, the discharge ($m^3/s$) is the product of the cross section area ($m^2$) and the average velocity of flow through average velocity of flow (m/s); so a decrease in cross section area will result in higher velocities which increases scour potential in the drain.

We can use this concept to assess the relative impact of various proposals. The sketch in Plate 4 illustrates changes to the surface levels within a floodway that might be considered as part of an open drain refurbishment and/or adjacent development. At first glance, the proposal may be considered acceptable, because the cross section area below the existing 5-year flood is increased due to the flatter batter slope on the LHS of the channel. Notwithstanding, the overall area below the 100-year flood is decreased due to the filling of land on the right hand side. In all likelihood, the works would result in increased flooding risk to upstream infrastructure during the 100-year ARI rainfall event.

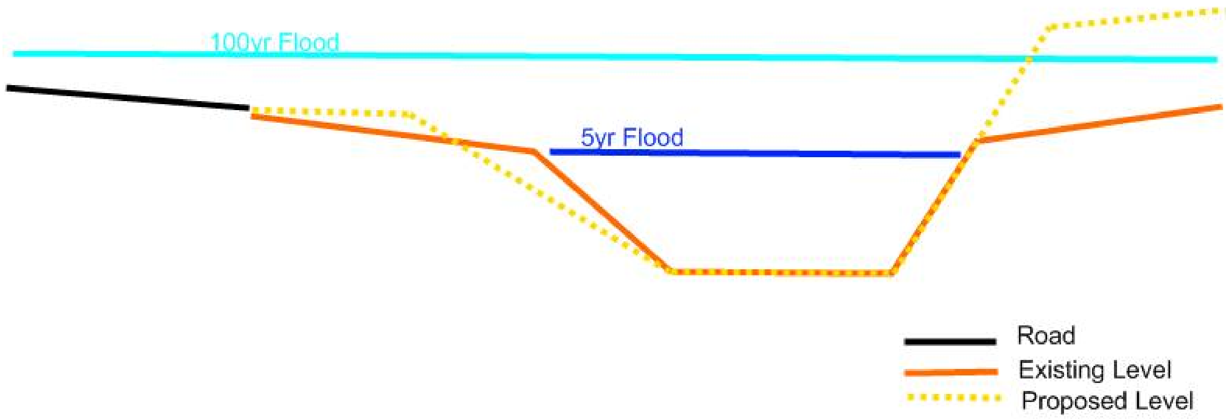



**Plate 4 – Effect of changes to drain cross section**

## 4.3 Culverts and bridges

Culverts and bridges at roads and for pedestrian access can be significant constraints to the capacity of the drainage network and often result the flooding of areas immediately upstream.

Conceptually, the cross section available for flow through the culvert is limited by the size and number of pipes. Additional energy losses occur as water enters the culvert, further reducing the relative capacity of the cross section. Plate 5 illustrates the potential impact of a culvert on flood risk upstream. When the capacity of the culvert is exceeded, the path will act as a weir and hold back floodwaters.

The velocity of flow through the culvert and over the path is higher than that of surrounding floodwaters as a result of the large difference in upstream and downstream water levels. Under certain conditions this can result in the formation of a hydraulic jump downstream of the culvert (as illustrated for the 100-year flood). These high velocities and unique hydraulic forces result in scour potential immediately upstream and downstream of the culvert and present can present safety issues for vehicles and/or pedestrians. Culvert headwalls and rock protection is commonly installed to mitigate the scour potential in these locations.

The culvert results in increased flood depth and reduces the average velocity in the upstream channel. The slower velocities allow some of the suspended material to settle out of the water column, this has been observed in South Hedland as silting of channels upstream of culverts. In this way, the presence of an appropriately designed culvert can assist in achieving environmental objectives.

Most bridges have abutments and deck levels which restrict the cross section of the channel and present similar issues for the hydraulic performance of the system. Notwithstanding, many of the existing footbridges that can be found across the town have relatively high decks and span the full width of the open channel. These bridges and have relatively low impact on the hydraulic capacity of the network.

Given the significant potential for flood impacts as a result of culverts, proposals to include culverts in a redevelopment of drainage reserves should be accompanied by a comprehensive hydraulic analysis by a suitably qualified drainage engineer.

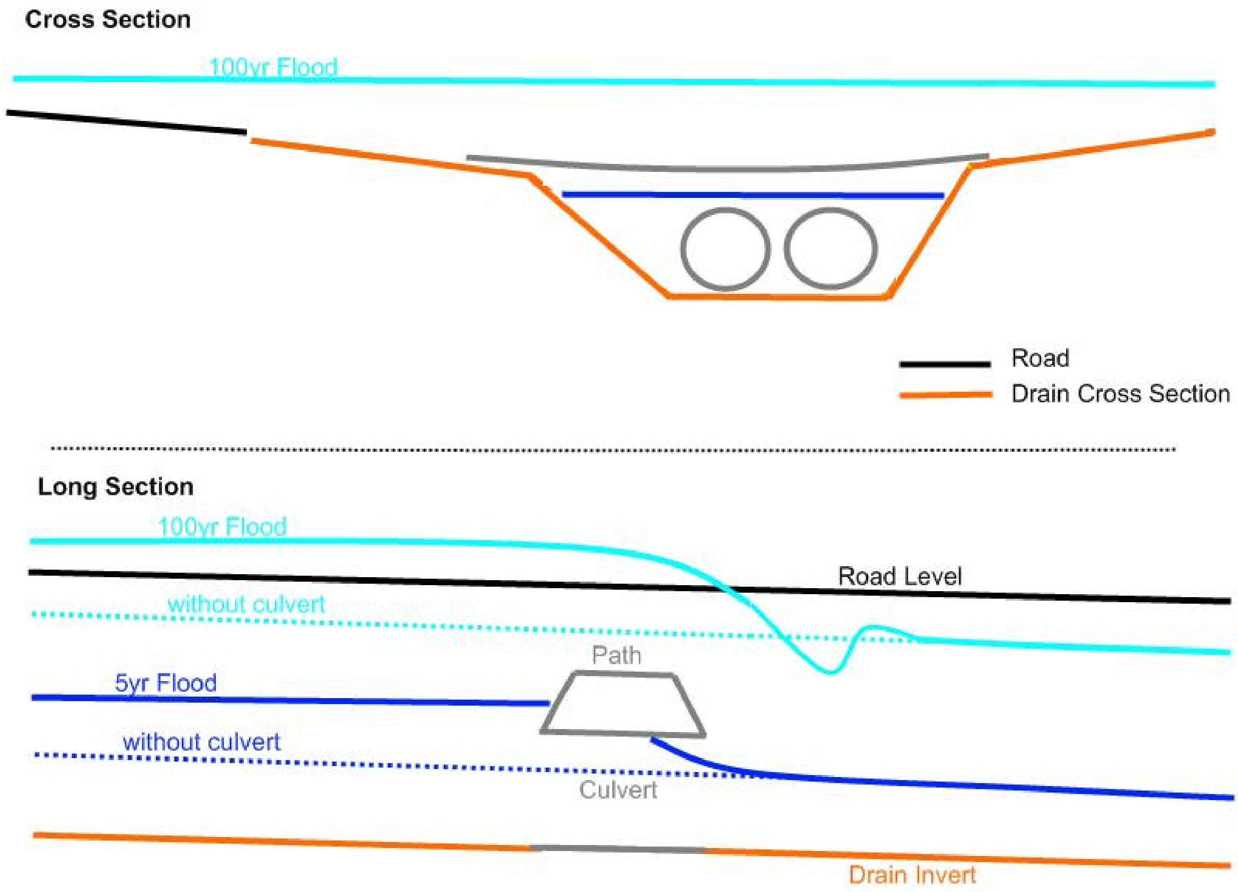



**Plate 5 – Impact of Culverts**

## 4.4 Multiple Use Areas

Multiple Use Areas are those which can provide a dual use. In terms of drainage reserves, a multiple use area might be a playing field or passive recreation area that is allowed to flood during storm events.

The redevelopment of drainage reserves as large areas of open space has potential to provide hydraulic benefits and better utilise drainage reserves. Temporary storage of flood water within these areas can reduce downstream discharge rates and presents opportunities for treatment for of storm water quality. The intensity of rainfall that occurs from Tropical Cyclones can reduce the effectiveness of detention areas for control of peak flows. Notwithstanding, where additional benefits can support their development, construction of multiple use areas could result in improvements to the hydraulic capacity of the drainage network.

Similar to the discussion in Section 4.2, hydraulic benefits are only afforded if the cross section for flow, or the storage volume for flood water is increased as a result of the development. The relative impact of upstream flooding and downstream flow rates depends on a number of parameters, including the timing and volume of stormwater coming from the upstream catchment and downstream hydraulic controls.

Finished levels and the location of community infrastructure need to be considered in the context of flood risk and the safety of users. Plate 6 illustrates a scenario whereby usable public open space can be constructed within a drainage reserve, whilst maintaining the drainage function for major events.

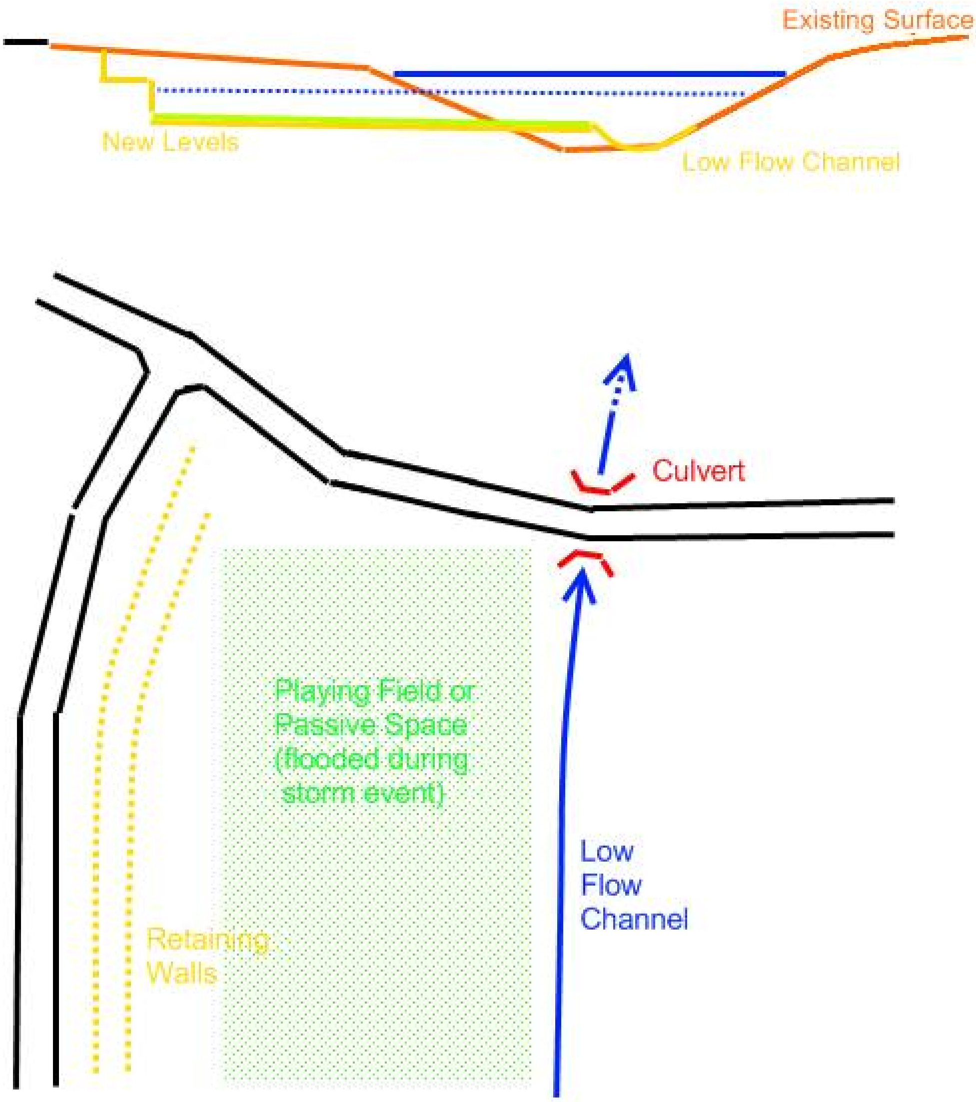



**Plate 6 – Multiple Use Area.**

## 4.5 Land Identified for development by the PLRP

The Public Land Rationalisation Plan identifies land parcels for redevelopment that are within or adjacent to existing drainage reserves. In accordance with the design criteria discussed in Section 3 of this report, floor levels of all new buildings should be set 300 mm above the peak 100-year ARI flood levels. Schedule 2 in Appendix B presents predicted flood levels for the existing and ultimate drainage networks as described earlier in the report.

Figures 5a to 5f illustrate that drainage reserves form major flow paths and should be maintained for drainage purposes to ensure that the predicted flood depths remain valid. Any development that has potential to obstruct the flows within the "floodway" should not be supported.

It is understood that Council has been presented with proposals to close existing drainage reserves in various parts of South Hedland. Specifically, it has been proposed to close drainage reserves:

1. between Brodie Crescent and Greene Place
2. from Acacia Way and Boronia Close through to Huxtable Cresent
3. from Somerset Crescent to Lawson Street
4. from Eucla Close to Delamere Street

These drainage reserves protect open drains and provide the network of flood paths that service existing property adjacent and upstream. Under no circumstances should any of these drainage reserves be closed without a comprehensive study that identifies a viable alternative drainage system, including a "floodway" (overland flow path) for large floods and that identifies the impact of the proposed changes on peak flood levels upstream. Any alternative flood path developed by relocation of existing drainage reserves should be protected by appropriate ownership and/or planning controls.

Notwithstanding, Council could consider that certain types of development do not present an obstruction to the major flow path, but should only consider such proposals in light of a comprehensive site specific hydraulic analysis to demonstrate potential impacts.

# 5. Hydrologic and Hydraulic Analysis

The existing and proposed drainage systems were modelled in Wallingford Software Ltd. Infoworks CS which calculates catchment hydrology performs and 1-dimensional hydraulic analysis to predict the performance of the drainage network. The predicted peak water levels from the 1-dimensional analysis was then analysed against a ground surface model to estimate the extent of flooding.

## 5.1 Catchment delineation and runoff parameters

Drainage survey information, contours, cadastre and site observations were used to define the extent of the catchments and delineate sub-catchments as illustrated in Figure 2 and Figures 5a to 5f.

The hydrologic model selected for this study considers an initial retention of rainfall before applying a proportional loss to estimate runoff. It is appropriate to consider the different responses that occur on impervious and pervious surfaces. Impervious surfaces include things such as roads, paved areas and roofs where runoff flows directly into the drainage system. Pervious surfaces include bare soil, gardens and road verges. The assumed parameters for both surfaces are presented in Table 2. The selected parameters reflect the dominant soil types in the study area consisting of fine sands and clay which have moderate infiltration capacity. The initial loss of 15 mm for pervious areas reflects localised ponding of water and storage within the catchment.

**Table 2 Runoff Parameters**

| Runoff Surface | Initial Loss (mm) | Proportional Loss (mm/mm) |
|---|---|---|
| Impervious | 1 | 0 |
| Pervious | 15 | 0.7 |

The ToPH Town Planning Scheme No 5 and the PLRP were used to characterise an ultimate land use scenario with three categories of land use as illustrated in Figure 3. For each land use the percentage of impervious area is estimated and presented in Table 3.

It is understood that a large portion of roof runoff in residential areas flows into gardens and other pervious areas and could be considered “disconnected” impervious areas. Notwithstanding, there may be significant runoff from residential land uses during the large rainfall events when onsite storage and infiltration capacities are exceeded. The actual proportion of roof area will vary for different residential densities. By example, roof area represents approx 20% for R10 lot (approx 1000m$^2$ lot area with a 200m$^2$ of roof area) and 40% for R20 lot (approx 500m2 lot area with 200m$^2$ of roof area). The adopted %Impervious (20%) for residential areas represents the indirect connection of roofs to the drainage system.

It should be noted that current theories regarding Water Sensitive Urban Design and protection of environmental assets consider that interception of stormwater on-site provides significant benefits for water quality and maintenance of natural water cycles. Therefore, while the selected parameters are considered appropriate to ensure a conservative assessment of flood risk, this should not be interpreted as a recommendation in regards to the need for on-site retention and infiltration of minor storm events.

The upstream catchment of South Creek was delineated and modelled as a rural catchment in consideration of tail water conditions for the drainage network. The proportional loss coefficient was reduced to 0.5 for the 100-year ARI to account for the different catchment response that occurs during large storm events. The total catchment area for South Creek at Great Northern Highway was estimated as 29 $km^2$, which is slightly larger than that assumed for the GPHSS; 23.2$km^2$.

**Table 3 Land Use Estimated Percentage Impervious**

| Land Use | Assumed Percentage Impervious |
|---|---|
| Commercial | 90% |
| Education | 20% |
| Public Open Space | 5% |
| Residential | 20% |
| Road Reserve | 60% |
| Rural | 0% |

## 5.2 Hydraulic Parameters and Assumptions

Drainage survey conducted as part of the study was used to establish representative dimensions of open drains and drainage structures throughout the study area.

Existing culvert dimensions and inverts were taken directly from the survey to establish the Schedule 1 in Appendix B. Photos of each culvert taken during the survey were used to estimate the percentage of the culvert height that is currently blocked by sediment and debris.

For each reach of open channel the survey cross sections were inspected to arrive at a representative trapezoid for an initial model run. The predicted floods from that initial run were observed and where constrained significantly as a result of the assumed trapezoidal cross section, the assumptions of those drains were reviewed to account for flood conveyance in adjacent drainage reserves.

The Hydraulic model was extended downstream to include parts of South Creek upstream of Great Northern Highway using the ground contours sourced from Landgate. The peak flow rates estimated by the runoff model are different to that estimated in the GPHSS, which estimated the peak flow rate during the 100-year ARI and 5-year ARI estimated as 383 $m^3$/s and 57 $m^3$/s respectively. Whilst the 5-year ARI peak flow predicted by this study is similar (54$m^3$/s) the 100-year peak flow is significantly different (162$m^3$/s). The first factor that contributes to this discrepancy is the significant volume of flood storage that is provided within South Hedland during the 100-year ARI event. The discussion in Section 5.3 provides further comment regarding the predicted flooding extent. The second factor is the validity of the method used in the GPHSS to estimate large flows for the catchment.

It is understood that the GPHSS used a revised Index Flood Method based on limited stream flow data to estimate peak flows for South Creek. Specific information regarding the composition of catchments for the “revised” method is not currently available. Appendix C documents a calculation of peak flows for South Creek using the ARR87 methods for the Pilbara Region which gives similar results to the method used in the GPHSS. The first observation is that the method is based on a limited dataset in which all catchments are much larger than that of South Creek. ARR87 reports that the data quality used to establish these generic methods for the Pilbara is poor and that flood estimates derived using those methods should be used with caution. Whilst the revised method used in the GPHSS is likely an improvement on the methods in ARR87, their accuracy in regards to a relatively small catchment such as South Creek remains unclear.

Further to the uncertainty described above, various studies have identified potential overflow between South Creek and South West Creek during large flood events. It is understood that the Town of Port Hedland in partnership with Landcorp have recently commissioned the Port Hedland Coastal Vulnerability Study that will supersede the outcomes of the previous flood modelling by GEMS. The accuracy and relevance of that study (to flooding in South Hedland) will depend on the methods and assumptions employed by the consultant.

To consider the impact of this uncertainty the model was modified to assume that during the 100-year ARI flood the catchment upstream of South Hedland generates sufficient peak flow to flood South Creek to the top-of bank. The topography is such that, even using this conservative assumption, at the locations that this “base flow” interacts with the end of the South Hedland Drainage channels, the relative water level is low enough such that there is no significant impact on the drainage network within South Hedland. Therefore, whilst GHD considers that the assumption may be overly conservative, there is no impact on the performance of the system.

## 5.3 Hydraulic Capacity of Existing Drainage Network

Peak flood water levels predicted for the existing drainage network under proposed land use scenario are presented in Schedule 2 (Appendix B).

These levels were used to predict the extent of potential flooding throughout the study area using survey of the drainage network and spot heights and surface contours sourced from Landgate. It should be noted that resolution of data available for areas outside of the drainage corridors limits the accuracy of the flooding extent. As such, the extent of flooding presented in flood maps is indicative and flood levels presented in Schedule 2 should be compared to site specific survey to assess flood risk to individual sites.

The 5-year ARI flood is generally contained within drainage reserves aside from flooding at some locations (Figure 4). The proposed drainage management strategy outlined in Section 3 seeks to contain the 5-year ARI event within the network of open drainage channels. Maintenance and upgrades required to achieve this objective are discussed in Section 6.

Predicted flood levels during the 100-year event result in significant areas of potential inundation. Without information as to the finished floor levels of buildings in these areas it is not possible to determine whether the potential flooding illustrated in the flood maps corresponds to actual flooding risk for those properties concerned.

Of particular concern is the potential for alternative flow paths to develop as drainage reserves fill and water flows through private properties and along road reserves. These overland flow paths transfer water between different parts of the network and reduce the accuracy of the result.

Further detailed studies using two dimensional flood modelling would be required to further refine the hydraulic model for the 100-year event. Such a model would account for alternative flow paths and the temporary storage of floodwaters within the flood plain, with the effect of reducing the predicted peak water levels in some parts of the study area. The modelling undertaken as part of this study can form the basis for development of a two dimensional flood model.

# 6. Drainage Network Improvements

The hydraulic modelling of the existing drainage network predicts flooding in number of areas where the predicted hydraulic capacity of the existing drainage system is not sufficient to convey the 5-year ARI flows without flooding surrounding areas.

## 6.1 Maintenance

As discussed in Section 2, sedimentation of drains and culverts and blockage of flow due to debris reduce the hydraulic capacity of the drainage network. Modelling suggests that by clearing culverts and regrading some key reaches of open drain to match existing culvert inverts will result in significant reduction in flooding throughout the town.

The hydraulic model was set up to simulate the effect of clearing all culverts and regrading the following sections of open drain.

- CN03 to CN09 including SH49;
- from SH26 through to SH29;
- from upstream of SH18 and from there through to SH19; and
- from upstream of SH2 and from there through to SX04.

Schedule 2 (Appendix B) presents new flood levels for those areas where there is a change as a result of the proposed maintenance.  Figures 5a to 5f illustrate the extent of flooding that could be expected after the proposed maintenance is complete. Of particular interest is that predicted flooding for the 5-year ARI has been significantly reduced or eliminated in the following areas:

- Bottlebrush Crescent near SH30.
- Boronia Close near SH49 (Plate 7).
- Egret Crescent and Spoonbill Crescent upstream of SH27.
- Steamer Avenue near SH18, and
- Roberts Street near SH2.


**Plate 7 – SH49 under Boronia Close**

## 6.2 Upgrades

Additional infrastructure upgrades will be required to improve the system performance further and to alleviate predicted flooding in the remaining areas. The recommended maintenance and improvements discussed below are proposed to address the key issues predicted during the 5-year ARI event.

Hydraulic modelling was performed to assess the overall performance of the proposed system after the improvements and maintenance regime is in place. Plates 8, 9, 12 and 13 illustrate the predicted flooding during the 5-year ARI after the proposed upgrades have been implemented.

The priority and cost of the recommended works are discussed further in Sections 6.3 and 6.4.

### 6.2.1 Site A – Parker Street

The predicted flooding near the intersection of Parker Street and Kennedy Street is due to the relatively low surface level at this location. Spot heights from Landgate suggest that road levels at the intersection are around 9.6m AHD.

During the critical 5-year ARI event, the Roberts Street drain floods back from SH1 (at North Circular Road) where the predicted peak water level is around 9.75m AHD. This results in a peak water level at Kennedy Street of 9.94m AHD. The invert of the drainage system is a major control to flooding and needs to be lowered in order to improve the capacity of the network. Unfortunately, there is limited scope to reduce the invert without extending the works all the way through to the outfall of the system.

Modelling suggests that lowering the invert of the open drainage system by 0.5m from the Kennedy / Parker Street intersection through to North Circular Road will reduce the predicted flooding during the 5-year ARI event (Plate 8).

These works would mean that for a large part of the drain, the invert will be below that of the culvert through North Circular Road (SH1) and could therefore be subjected to increased frequency of sedimentation and periods of inundation. Due to these maintenance and public nuisance issues Council may like to consider additional works to regrade SH1 and the downstream channel. The need for those additional works should be considered after inspecting the performance of the system after initial works have been implemented. As such the upgrade of SH1 has not been included upgrade plan.

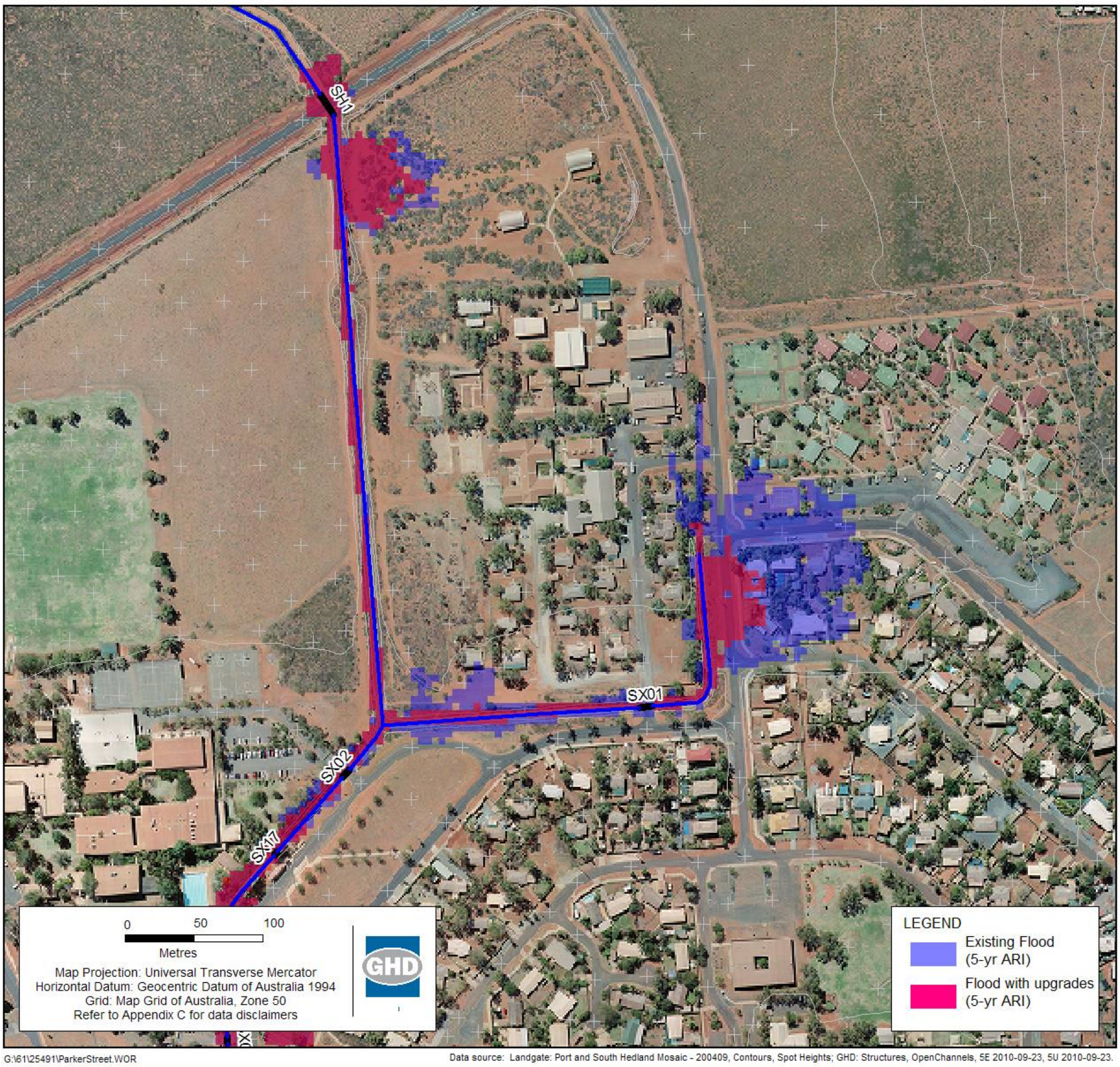



**Plate 8 - Parker Street Flooding**

### 6.2.2 Site B – Traine Crescent and Edkins Place

The culverts at Brodie Crescent (SH60 - 4 x 450mm dia RCPs) are insufficient to convey peak flows during the critical 5-year ARI rainfall event and result in flooding upstream and overtopping of the road near the intersection of Driver Way. The flooding shown at Traine Crescent and Edkins Place represents the potential flow path for water overflowing Brodie Crescent.

Upgrading SH60 to 3 x 1200mm x 600mm box culverts will provide sufficient capacity to convey the predicted 5-year ARI flow without overtopping and reduces the predicted extent of upstream flooding as illustrated in Plate 9.

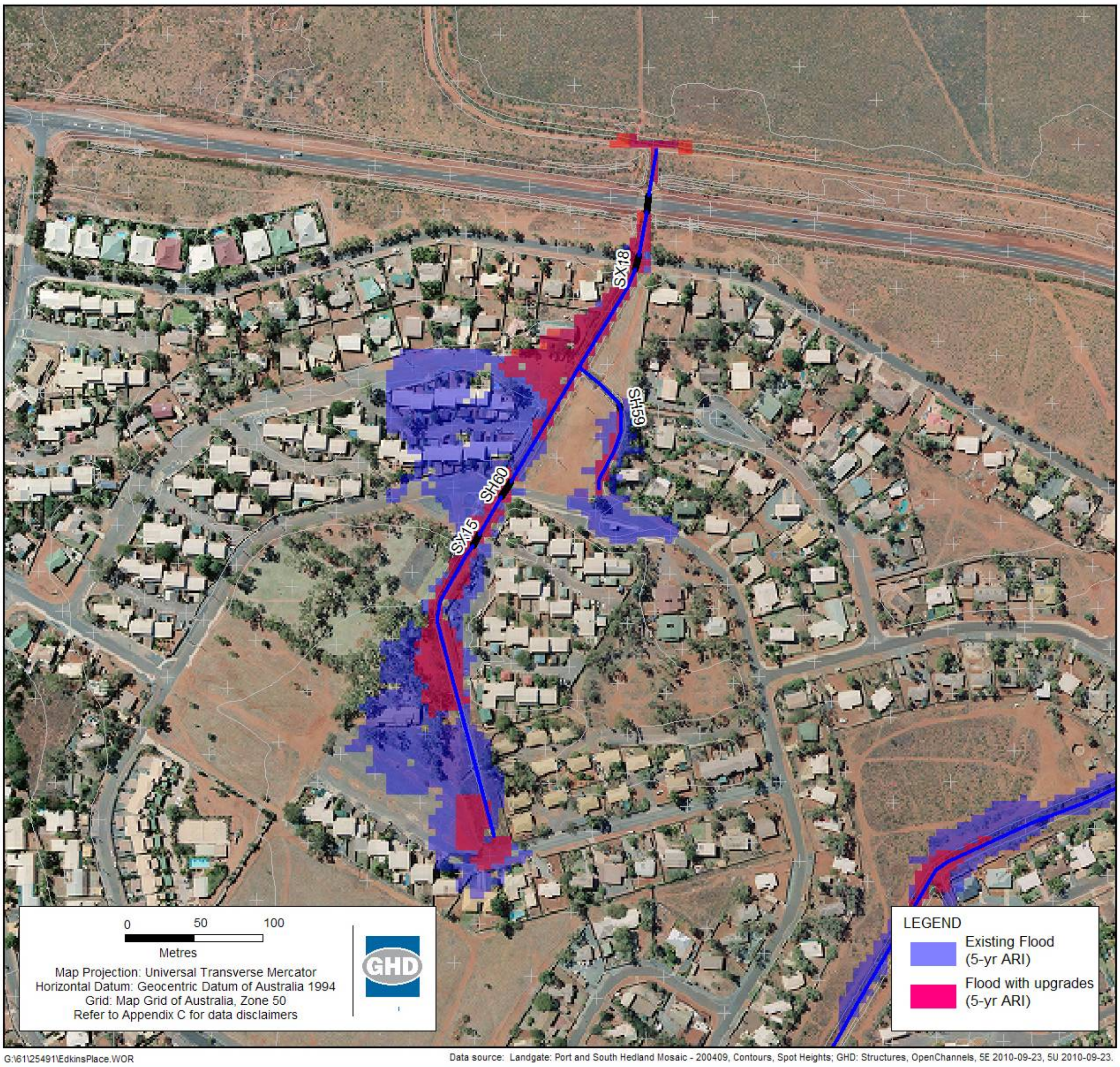



**Plate 9 – Edkins Place Flooding**


**Plate 10 – SH60 under Brodie Crescent**

### 6.2.3 Site C – Brodie Crescent & Draper Place

The predicted flooding near around Brodie Crescent and Draper Place is due to the relatively low surface level at this location. Spot heights from Landgate indicated levels on both roads around 10.3m AHD in contrast to the surrounding area which is around 11 m AHD.

During the critical 5-year ARI event, predicted peak water level in the drain downstream of Brodie Crescent is around 9.47m AHD. The culvert under Brodie Crescent (SH52) results in additional head losses and generates a peak water level upstream at 10.66m AHD.


**Plate 11 – SH52 under Brodie Crescent**

Modelling suggests that regrading and redefining the open drainage system between Dale Street and Paton Road could help to reduce the severity of flooding, reducing the peak water level at upstream of Brodie Crescent to 10.35m AHD.  Unfortunately, the new open drain would be near level and highly susceptible to sedimentation.

As an alternative it is recommended that Council consider construction of an attenuation basin between Cottier Drive and Brodie Crescent and the upgrade of the culvert SH52. Modelling suggests that a basin with a top area of approx 1 ha, and upgrade of SH52 to have 2 x 1200mm x 750mm box culverts would reduce tailwater sufficiently such that peak water levels upstream do not exceed 10.27m AHD during the critical 5-year ARI storm. This option is illustrated in Plate 12.

Another potential solution is to establish an alternate drainage path via a connection to the existing drain on the south side of Cottier Drive (near SX13) where the predicted 5-year ARI peak water levels are around 10.1m AHD. Further modelling and detailed design would be required to confirm the viability of this alternative.

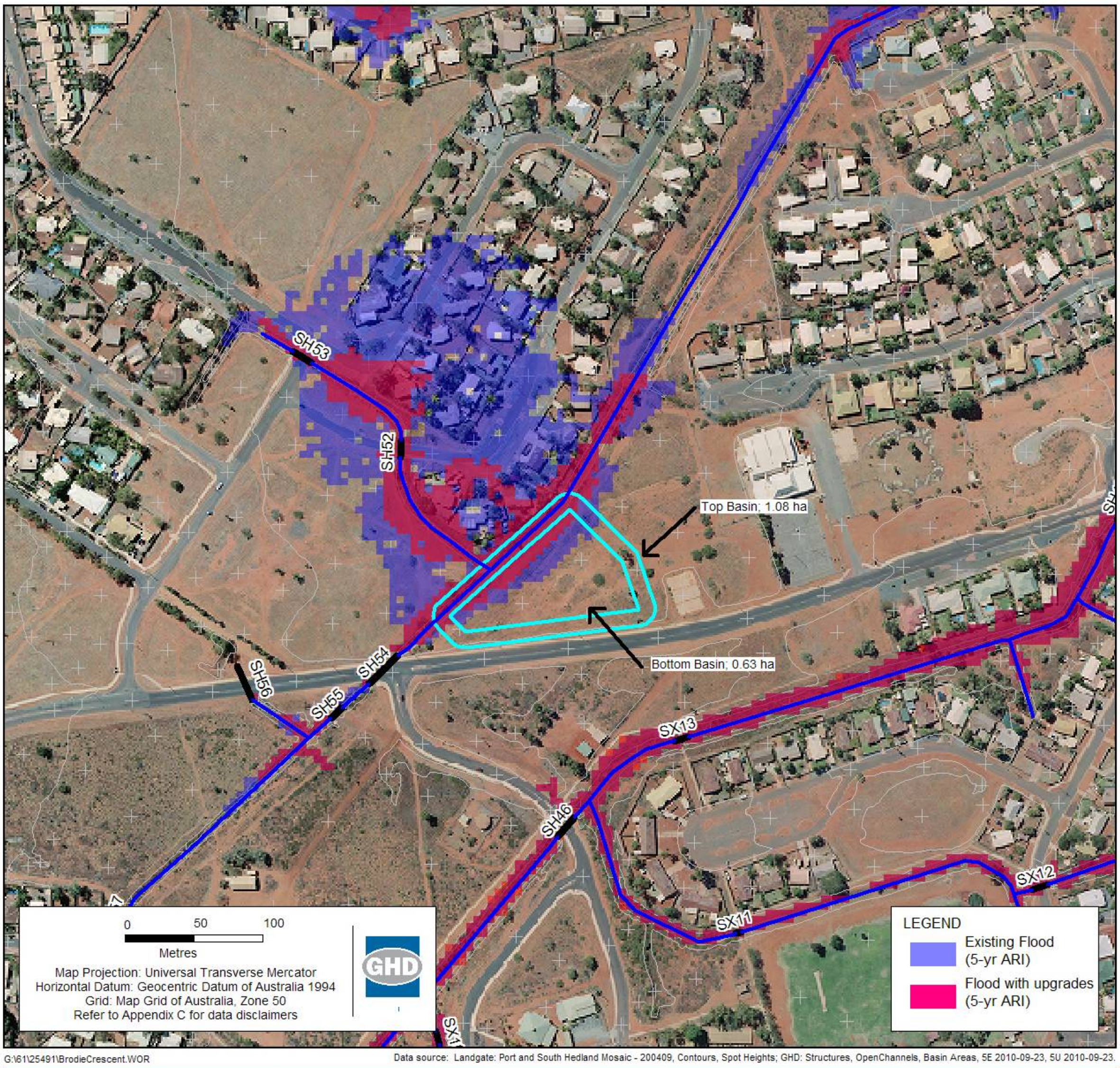



**Plate 12 - Brodie Crescent Flooding**

### 6.2.4 Site D – Acacia Way

The survey observed a culvert (SH50) that appears to be installed to provide construction access to an adjacent site. The observed culvert (SH50) comprising a single 400mm PVC pipe is insufficient to convey peak flows during the critical 5-year ARI rainfall event and will result in flooding to Acacia Way.

SH50 should be removed in order to reduce the flooding risk in the surrounding area. A permanent crossing would require modifications to the inverts of the open drain to improve the hydraulic capacity as illustrated in Plate 13.

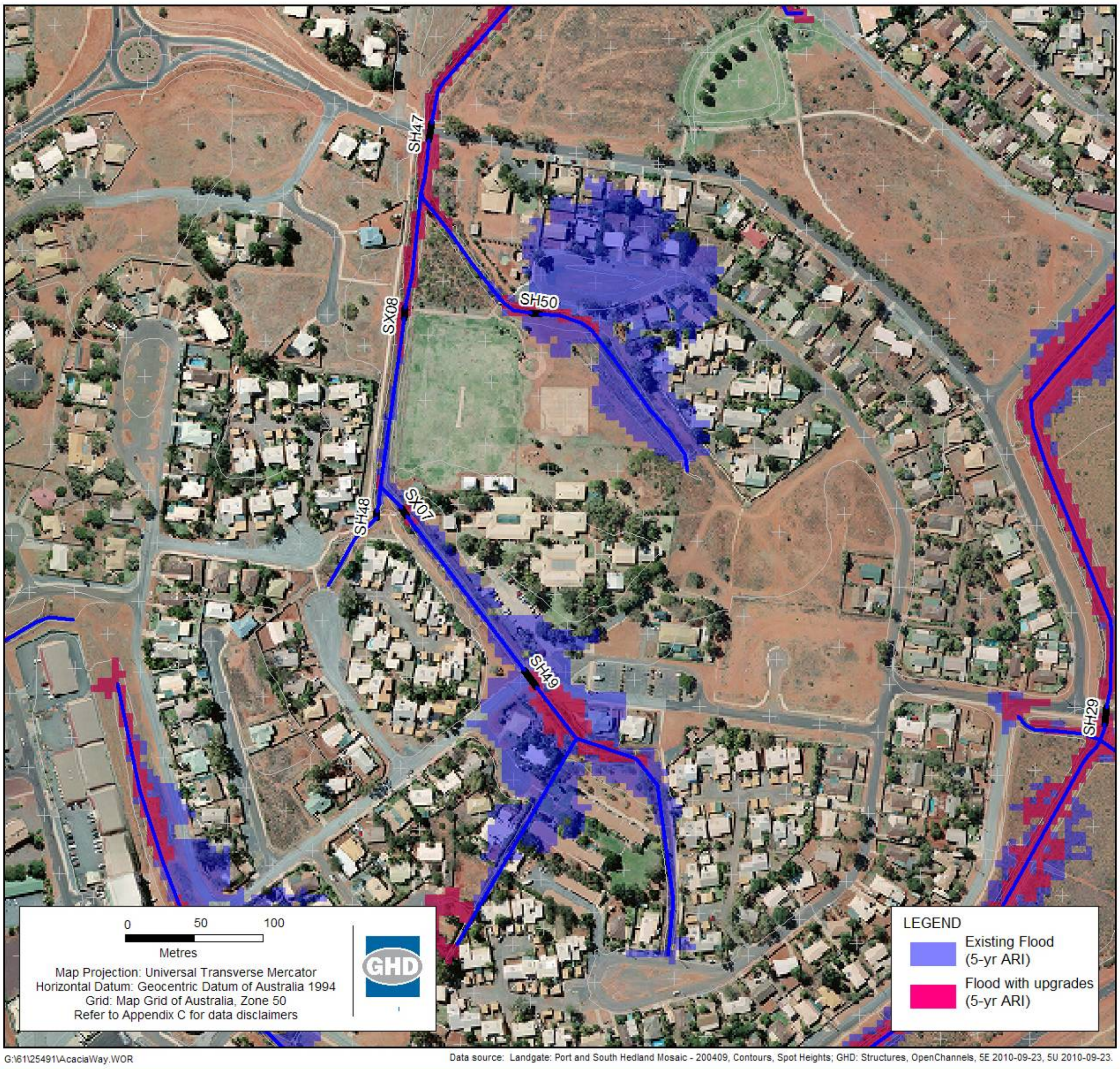



**Plate 13 - Acacia Way Flooding**


**Plate 14 – SH50 near Acacia Way**

### 6.2.5 Site E – Gascoyne Court

The flooding illustrated around Gascoyne Court is largely misleading and results from the density of the spot height data that is available and the analysis technique used to produce flood maps. Spot heights from Landgate suggest that the road levels at the Gascoyne Court cul-de-sac is approximately 12.6m AHD which is above the predicted peak 5-year ARI water level of 12.44m AHD upstream of PE21.

The more detailed flood modelling proposed as per Section 5.3 would provide more accurate flood mapping in this area.

## 6.3 Priority of works

The priority of proposed maintenance and upgrades should be determined by the relative risk to property damage or injury. It is recommended that survey of finished floor levels for existing buildings and more reliable ground survey is obtained in the those areas to allow more detailed modelling of flood extent and to assess the risk of flooding to individual properties.

The flood modelling predicts large areas of inundation as a result of a 100-year ARI rainfall event. Given the extent of the flooding predicted by this study Council may like to consider undertaking additional flood modelling using a two dimensional model.

The proposed maintenance and upgrades of the drainage network aim to address predicted flooding in a 5-year ARI rainfall event. Considering the number of properties affected as a measure of priority, the key works identified as part of this study can be prioritised as per Table 4.

**Table 4 Priority of works**

| Project | Number of Affected Properties | Works Description | Location of Flooding |
|---|---|---|---|
| 1 | 20 | Construct basin and upgrade culvert under Brodie Crescent (SH52). | Brodie Crescent / Draper Place |
| 2 | 19 | Upgrade culvert under Brodie Crescent (SH60). | Traine Crescent / Edkins Place |
| 3 | 13 | Remove temporary culvert SH50. | Acacia Way |
| 4 | 10 | Regrade drain and clean culverts from SH26 through to SH29. | Egret Crescent / Spoonbill Crescent |
| 5 | 9 | Regrade drain and clean culverts upstream of SH18 to SH19. | Steamer Avenue |
| 6 | 7 | Regrade drain from N049 through to SH1. | Parker Street |
| 7 | 6 | Regrade drain from N033 to NA55, clean culvert under Boronia Close (SH49). | Boronia Close |
| 8 | 6 | Regrade drain and clean culverts from upstream of SH2 to SX04. | Roberts Street |
| 9 | 5 | Clean culvert under Gregory Street (SH30), localised regrading | Bottlebrush Crescent |

The number of affected properties in Table 4 was determined by inspecting the aerial photography from 2004 and the indicative extent of flooding for the 5-year ARI rainfall event. As such, the relative priority is indicative and additional data regarding finished levels of properties and buildings should be considered to confirm the extent to which properties are affected.

## 6.4 Cost estimates

The cost and accuracy of a two dimensional flood model will depend on the availability of detailed ground survey. To improve on the accuracy of the flood modelling undertaken as part of this study it is recommended that two dimensional flood model is only developed if more accurate survey such as laser survey (LIDAR) is available. It is understood that Landcorp have recently commissioned the Port Hedland Coastal Vulnerability Study which may have involved collection of LIDAR for South Hedland. The ToPH should expect consultant fees for preparation of flood maps using a two dimensional modelling between $50,000 and $60,000.

Cost estimates for each of the projects identified above have been prepared to provide some indication as to the relative magnitude of each project and are presented in Table 5. It should be noted that the cost of the proposed works will be dependant on detailed design of infrastructure. Survey of the existing drainage infrastructure can be used to refine quantities and cost estimates during detailed design.

GHD has prepared these cost estimates using information from Rawlinsons Australian Construction Handbook and where required, based on assumptions made by GHD. Prices and quantities in the cost estimate may change. GHD does not represent, warrant or guarantee that the project can be completed for the cost estimates prepared by GHD. In regards to the estimates:

- An allowance has been made for cartage of spoil within 10km of each site.
- No allowance has been made for landscaping of areas after excavation.

**Table 5 Cost Estimates**

| Project | Cost Basis | Estimate |
|---|---|---|
| 1 | 16200 m3 excavation, replace 13m culvert with 2 x 1200 mm x 750 mm RCBC. | $ 459,400 |
| 2 | Replace existing 13m culvert with 3 x 1200 mm x 600 mm RCBC. | $ 91,600 |
| 3 | Redefine drain; 500 $m^3$ excavation (nominal) | $ 11,300 |
| 4 | 5500 $m^3$ excavation (8$m^3$ x 680m) | $ 122,400 |
| 5 | 2700 $m^3$ excavation (8$m^3$ x 340m) | $ 77,700 |
| 6 | 7800 $m^3$ excavation (10$m^3$ x 780m) | $ 224,100 |
| 7 | 3600 $m^3$ excavation (8$m^3$ x 450m) | $ 103,400 |
| 8 | 3900 $m^3$ excavation (8$m^3$ x 490m) | $ 112,200 |
| 9 | 300$m^3$ excavation (nominal). | $ 8,600 |

## 6.5 Opportunities and Potential Funding Sources

Council could consider a number of funding sources and opportunities in regards to the upgrade of drainage networks. Most of the works proposed above are the result of long term maintenance deficiencies and Council may consider that it is appropriate to fund those works from municipal funds.

Notwithstanding, some of the proposed upgrades can be associated with recent and future developments. Council could consider a levy on developments drainage catchments that contribute to the need for upgrades. By example, the severity of flooding associated with “Project 4” will be impacted by the development of land south of additional discharge from development of land south of Osprey Drive. Similarly, the recent and proposed development around Traine Crescent contributes to the need for upgrades in the area.

The proposed construction of a detention basin on Cottier Drive presents an opportunity to provide a drainage function as part of a multiple use area. In this way there may be an opportunity to partially offset drainage upgrades as part of a larger redevelopment of public open space in the area.

# 7. Asset Management and Maintenance

## 7.1 Background

The global definition for asset management is "the optimal lifecycle management of physical assets to sustainably achieve the stated business objectives" according the European Federation for National Maintenance Societies (EFNMS).

In the case of the South Hedland drainage network, the drainage infrastructure and drainage reserves represent assets, whilst business objectives are defined by the effective operation of the system.

This part of the drainage study aims to identify key assets in the drainage network and provide management recommendations that will ensure the future operational efficiency of the drainage network through scheduled maintenance.

## 7.2 Existing Condition

The drainage network is constructed consistently throughout the town site which will facilitate efficient maintenance. This drainage study has identified the drainage network is relatively good condition aside from some key drainage issues and blockages as discussed in Section 6. The main issues that have been identified relate to the accumulation of sediment and blockage of culverts. In new development areas where the drainage network was recently constructed, there is evidence of scour on the banks of some drainage channels, highlighting the need for vegetation to stabilise these slopes.

Given the nature of the drainage network, vegetation management should also be a maintenance consideration. By example, overgrown vegetation at kerb breaks can impinge on flows and cause localised flooding.

## 7.3 Maintenance Schedule

Given nature of the drainage system, it is recommended that Council undertake two inspections of the drainage network each year to identify maintenance works required. This schedule will enable Council to identify critical maintenance issues in a timely manner and allow for ensure long term viability of key drainage infrastructure. The maps and GIS information developed as part of this study can be used to guide future on ground inspections and maintenance.

The first inspection should occur after the wet season (around April or May) to identify the need for any major works due to sedimentation or damage during the year past. A bi-annual survey of inverts in open drains should be undertaken as part of this inspection to help identify accumulation of sediment over the longer term, identifying key maintenance issues such as "Project 4" through "Project 9" identified above.

A second inspection should be undertaken later in the year, to prompt the removal of debris and materials that have potential to cause obstructions to culverts or impact on the conveyance of floodwater in open drains. It is understood that the ToPH promote an annual "pre-cyclone clean up" for waste collection. It is recommended that this program be maintained and extended to address maintenance of drainage infrastructure.

The proposed maintenance schedule is presented below in Table 6.

**Table 6 Drainage maintenance schedule**

| Inspection Timing | Inspection Type | Resulting Maintenance |
|---|---|---|
| After major events | Inspection for damages i.e. structural damages to culverts | Repair of damages to structures |
| End Wet Season (April / May) | Inspection for blockages / sedimentation to culvert, drains and other infrastructure | Removal of blockages / sediment during dry season |
| End Dry Season (Sept / Oct) | Inspection for foreign items, dumped goods | Removal of foreign items |
| Every 2 years | Survey drain long sections to identify accumulation of sediment | Regrade open drains. |
| Every 5 years | Inspection for overgrown vegetation | Vegetation maintenance, maintain grasses < 100 mm |

## 7.4 Vegetation Maintenance

Maintaining a level of vegetation cover in constructed features such as drainage reserves is important in increasing soil stabilisation and reducing erosion. Conversely, vegetation can act to inhibit stormwater flows and lead to localised flooding or in more serious cases, reduce the capacity of open drains. A balance of vegetation cover is required to ensure the optimal drainage system performance.

The study identified significant issues from sedimentation of drainage infrastructure. Further, recently developed and redefined drainage channels will be subject to accelerated bank erosion. It is therefore recommended that where possible, vegetation is established in drainage channels and drainage reserves. Some established drainage channels are covered in native grasses, with small shrubs and trees occupy the drainage reserves. This level of vegetation cover appears to be sustainable and could significantly reduce erosion and sedimentation within the drainage network.

Notwithstanding the maintenance schedule presented in Section 7.3 and the recommendations contained within Section 6, common sense should prevail when conducting maintenance around existing vegetation. By applying flexibility in the desired cross section during maintenance works some existing vegetation can be retained significantly reducing the likelihood of future sedimentation.

Appendix A

# Figures

**Figure 1 – IPWEA, Floodplain Development Strategy**

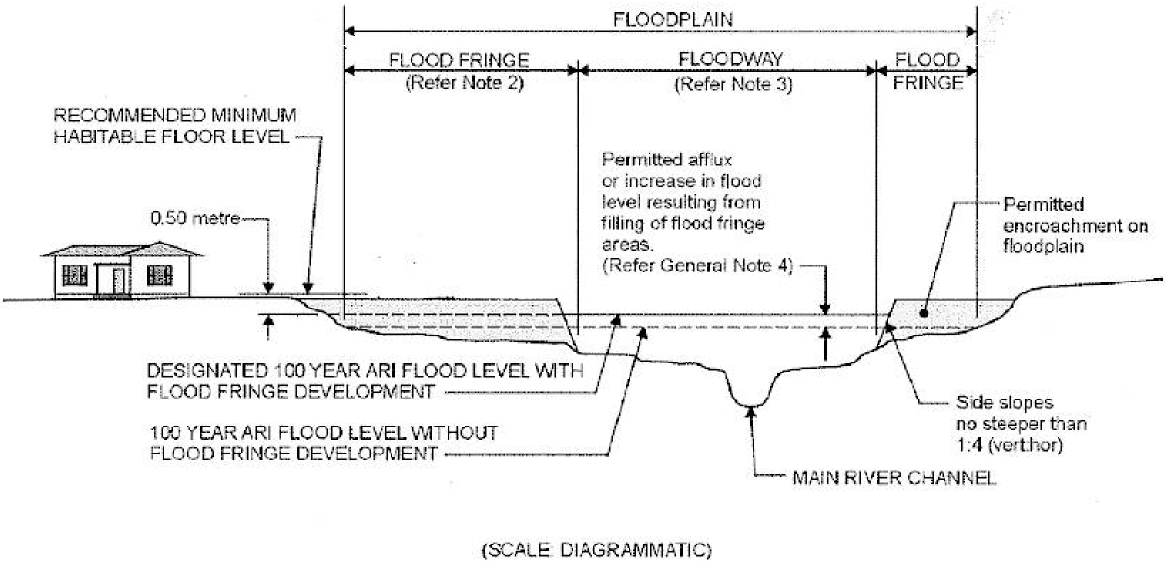
**Figure 2 – Topography and Catchment delineation**

**Figure 3 – Land Use Delineation for Flood Modelling**

**Figure 4 – Flooding from Existing Scenario**

**Figures 5a to 5f – Flooding after Maintenance**

# RECOMMENDED FLOODPLAIN DEVELOPMENT STRATEGY




GENERAL NOTES

1. The 100 year ARI flood level is expected to occur, on average, once every 100 years. Floods higher than this level will occur but will be less frequent.

2. The flood fringe is an area affected by a 100 year ARI flood. Development (ie, filling, building, etc) that is located within the *flood fringe* is considered acceptable with respect to major flooding. However, a minimum habitable floor level of 0.50 metre above the adjacent 100 year ARI flood level is recommended to ensure adequate flood protection.

3. Development (ie, filling, building, etc) that is located within the *floodway* and is considered obstructive to major flows is not acceptable as it would increase flood levels upstream. No new dwellings are acceptable within the floodway.

4. The increase in flood level that will result from total development of flood fringe areas has been calculated to be no greater than 0.15 metre.

5. A failure to properly adhere to these recommendations will result in a greater exposure to risks of flood damage.

**FIGURE 4.3: FOODPLAIN DEVELOPMENT STRATEGY**

*SOURCE: Department of Water*

**Figure 1 – IPWEA, Floodplain Development Strategy**

Figure 2 – Topography and Catchment delineation

Figure 3 – Land Use Delineation for Flood Modelling

Figure 4 – Flooding from Existing Scenario

Figures 5a to 5f – Flooding after Maintenance

Appendix B

# Schedules

## Schedule 1 – Existing Structures

| ID | Type | Width (mm) | Height (mm) | No Barrels | US Invert | DS Invert | Current Blockage % |
|---|---|---|---|---|---|---|---|
| SH1 | Box Culvert | 1800 | 900 | 3 | 8.32 | 8.4 | 15 |
| SH2 | Box Culvert | 1200 | 600 | 3 | 10.06 | 10.31 | 50 |
| SH3 | Box Culvert | 4500 | 2150 | 2 | 9.24 | 9.24 | 10 |
| SH4 | Box Culvert | 1200 | 900 | 4 | 9.4 | 9.26 | 35 |
| SH6 | Box Culvert | 1200 | 750 | 2 | 10.26 | 10.16 | 10 |
| SH7 | Box Culvert | 1200 | 720 | 2 | 10.75 | 10.67 | 10 |
| SH9 | Box Culvert | 1200 | 900 | 2 | 11.04 | 10.89 | 10 |
| SH11 | Box Culvert | 1200 | 900 | 2 | 11.5 | 11.42 | 10 |
| SH12 | Box Culvert | 900 | 870 | 2 | 11.77 | 11.67 | 10 |
| SH13 | Pipe Culvert | 920 | | 2 | 11.98 | 11.9 | 30 |
| SH15 | Box Culvert | 600 | 600 | 2 | 12.57 | 12.48 | 20 |
| SH16 | Box Culvert | 750 | 450 | 2 | 12.72 | 12.66 | 15 |
| SH17 | Box Culvert | 900 | 750 | 2 | 11.52 | 11.3 | 50 |
| SH18 | Box Culvert | 900 | 450 | 1 | 12.81 | 12.68 | 80 |
| SH19 | Pipe Culvert | 750 | | 2 | 12.18 | 12.04 | 20 |
| SH20 | Box Culvert | 1200 | 600 | 2 | 11.7 | 11.62 | 50 |
| SH21 | Box Culvert | 900 | 900 | 2 | 11.43 | 11.24 | 20 |
| SH22 | Box Culvert | 1200 | 600 | 1 | 12.9 | 12.82 | 50 |
| SH23 | Box Culvert | 1200 | 600 | 1 | 12.54 | 12.4 | 50 |
| SH24 | Pipe Culvert | 900 | | 2 | 11.9 | 11.85 | 15 |
| SH25 | Box Culvert | 1200 | 750 | 2 | 11.07 | 10.89 | 20 |
| SH26 | Box Culvert | 1200 | 900 | 4 | 11.45 | 11.37 | 60 |
| SH27 | Box Culvert | 1200 | 900 | 5 | 11 | 10.93 | 50 |
| SH28 | Box Culvert | 1200 | 1200 | 5 | 10.81 | 10.8 | 30 |
| SH29 | Box Culvert | 1830 | 1200 | 3 | 10.43 | 10.32 | 20 |
| SH30 | Pipe Culvert | 360 | | 2 | 11.52 | 11.46 | 70 |
| SH31 | Box Culvert | 1200 | 750 | 1 | 11.1 | 11.14 | 20 |
| SH32 | Box Culvert | 1530 | 1200 | 3 | 9.89 | 9.69 | 30 |

| ID | Type | Width (mm) | Height (mm) | No Barrels | US Invert | DS Invert | Current Blockage % |
|---|---|---|---|---|---|---|---|
| SH34 | Box Culvert | 1200 | 900 | 5 | 9.69 | 9.65 | 25 |
| SH36 | Box Culvert | 1800 | 900 | 3 | 9.13 | 9.13 | 15 |
| SH37 | Pipe Culvert | 910 | | 4 | 10.41 | 10.22 | 40 |
| SH42 | Box Culvert | 1800 | 900 | 3 | 8.63 | 8.65 | 35 |
| SH43 | Box Culvert | 1800 | 1200 | 3 | 8.48 | 8.42 | 55 |
| SH44 | Box Culvert | 1800 | 1200 | 3 | 8.59 | 8.57 | 30 |
| SH45 | Box Culvert | 1800 | 1200 | 2 | 9.15 | 8.96 | 15 |
| SH46 | Box Culvert | 1200 | 900 | 2 | 9.86 | 9.74 | 15 |
| SH47 | Box Culvert | 1200 | 900 | 2 | 9.96 | 9.98 | 30 |
| SH48 | Box Culvert | 910 | 440 | 1 | 11.38 | 11.37 | 25 |
| SH49 | Box Culvert | 910 | 750 | 1 | 10.48 | 10.44 | 85 |
| SH50 | Pipe Culvert | 400 | | 1 | 10.57 | 10.8 | 5 |
| SH51 | Box Culvert | 1200 | 900 | 5 | 9.36 | 9.52 | 15 |
| SH52 | Box Culvert | 1200 | 750 | 1 | 9.54 | 9.42 | 20 |
| SH53 | Box Culvert | 1220 | 780 | 1 | 9.77 | 9.68 | 30 |
| SH54 | Box Culvert | 900 | 900 | 2 | 9.75 | 9.76 | 15 |
| SH55 | Box Culvert | 880 | 900 | 2 | 9.7 | 9.67 | 15 |
| SH56 | Box Culvert | 900 | 450 | 1 | 10.1 | 10.04 | 40 |
| SH57 | Pipe Culvert | 500 | | 2 | 10.46 | 10.41 | 65 |
| SH58 | Box Culvert | 1200 | 450 | 5 | 8.55 | 8.5 | 50 |
| SH59 | Box Culvert | 1200 | 900 | 1 | 9.43 | 9.43 | 25 |
| SH60 | Pipe Culvert | 450 | | 4 | 9.6 | 9.6 | 0 |
| SX01 | Bridge | 6500 | 1240 | | | | |
| SX02 | Bridge | 5900 | 1080 | | | | |
| SX03 | Bridge | 8500 | 1200 | | | | |
| SX04 | Bridge | 8300 | 1470 | | | | |
| SX05 | Foot Bridge | 10300 | 1740 | | | | |
| SX06 | Foot Bridge | 9380 | 1690 | | | | |
| SX07 | Foot Bridge | 10300 | 1030 | | | | |
| SX08 | Foot Bridge | 8200 | 1510 | | | | |
| SX09 | Foot Bridge | 11100 | 1460 | | | | |
| SX10 | Foot Bridge | 8200 | 1430 | | | | |

| ID | Type | Width (mm) | Height (mm) | No Barrels | US Invert | DS Invert | Current Blockage % |
|---|---|---|---|---|---|---|---|
| SX11 | Foot Bridge | 8800 | 990 | | | | |
| SX12 | Box Culvert | 900 | 600 | 0 | 9.98 | 9.9 | 0 |
| SX13 | Foot Bridge | 11690 | 1390 | | | | |
| SX14 | Bridge | 6700 | 1390 | | | | |
| SX15 | Bridge | 7300 | 1050 | | | | |
| SX16 | Pipe Culvert | 600 | | 2 | 12.4 | 12.31 | 0 |
| SX17 | Foot Bridge | 5900 | 970 | | | | |
| SX18 | Box Culvert | 1200 | 1200 | 5 | 8.58 | 8.55 | 0 |
| SX19 | Bridge | 6100 | 1150 | | | | |

## Schedule 2 - Predicted Upstream Flood Levels

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel / Structure** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| CN01 | 11.99 | 11.88 | 12.15 | 12.04 |
| CN02 | 12.25 | | 12.39 | 12.37 |
| CN03 | 11.98 | 11.44 | 12.06 | 11.71 |
| CN04 | 11.14 | 10.83 | 11.50 | 11.20 |
| CN05 | 11.91 | | 12.03 | |
| CN06 | 11.35 | | 11.44 | 11.39 |
| CN07 | 11.03 | 10.76 | 11.39 | 11.18 |
| CN08 | 11.00 | 10.74 | 11.38 | 11.18 |
| CN09 | 10.69 | 10.62 | 11.29 | 11.15 |
| CN11 | 11.55 | | 11.69 | |
| CN12 | 10.90 | 10.72 | 11.28 | 11.14 |
| CN13 | 10.56 | 10.58 | 11.28 | 11.14 |
| CN14 | 10.52 | 10.55 | 11.07 | 11.01 |
| CN15 | 10.43 | 10.46 | 11.05 | 10.98 |
| CN16 | 10.98 | | 11.08 | 11.07 |
| CN17 | 10.56 | | 11.05 | 10.96 |
| CN18 | 10.39 | 10.40 | 11.04 | 10.96 |
| CN19 | 10.26 | 10.29 | 10.96 | 10.81 |
| CN20 | 10.89 | | 11.09 | |
| CN21 | 10.76 | | 11.10 | |
| CN22 | 10.75 | | 11.00 | |
| CN23 | 10.75 | | 11.00 | |
| CN24 | 10.52 | | 10.97 | 10.84 |
| CN25 | 10.24 | 10.26 | 10.96 | 10.80 |
| CN26 | 10.07 | 10.08 | 10.95 | 10.76 |
| CN27 | 9.97 | 9.96 | 10.92 | 10.71 |
| CN28 | 9.97 | 9.96 | 10.92 | 10.71 |
| CN29 | 10.28 | | 10.93 | 10.71 |
| CN30 | 9.96 | 9.95 | 10.92 | 10.71 |
| CN31 | 9.94 | 9.93 | 10.90 | 10.64 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel / Structure** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| CN32 | 10.12 | | 10.85 | 10.58 |
| CN33 | 9.84 | 9.78 | 10.84 | 10.54 |
| CN34 | 9.77 | | 10.62 | 10.50 |
| CN35 | 9.60 | 9.61 | 10.35 | |
| CN36 | 9.59 | 9.60 | 10.35 | 10.34 |
| GE01 | 10.68 | | 11.06 | 11.05 |
| GE02 | 10.66 | | 10.90 | 10.89 |
| GE03 | 10.65 | | 10.86 | 10.85 |
| GE04 | 9.91 | 9.90 | 10.55 | 10.42 |
| GE05 | 10.22 | 10.13 | 10.57 | 10.44 |
| GE06 | 9.73 | 9.71 | 10.49 | 10.37 |
| GE07 | 9.71 | 9.68 | 10.49 | 10.36 |
| GE08 | 9.16 | | 9.62 | |
| GE10 | 9.10 | | 9.55 | 9.56 |
| GE11 | 9.05 | | 9.50 | 9.51 |
| MT01 | 13.70 | | 14.32 | 14.06 |
| MT02 | 13.46 | | 14.32 | 14.06 |
| MT03 | 12.98 | 12.99 | 13.86 | 13.46 |
| MT04 | 13.34 | | 13.86 | 13.47 |
| MT05 | 12.84 | | 13.86 | 13.45 |
| MT06 | 12.82 | | 13.86 | 13.45 |
| MT07 | 12.11 | 11.88 | 13.15 | 13.08 |
| MT08 | 11.24 | 11.35 | 11.81 | 11.83 |
| PA01 | 10.81 | 10.79 | 11.42 | 11.41 |
| PA02 | 10.73 | 10.68 | 11.41 | 11.39 |
| PA03 | 10.70 | 10.64 | 11.40 | 11.38 |
| PA04 | 10.59 | 10.55 | 11.24 | 11.21 |
| PA05 | 10.46 | | 10.94 | 10.93 |
| PA07 | 10.81 | 10.70 | 11.35 | 11.34 |
| PA08 | 10.66 | 10.60 | 10.97 | |
| PA09 | 10.47 | | 10.95 | 10.94 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel / Structure** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| PA10 | 10.44 | | 10.93 | 10.92 |
| PA11 | 10.21 | | 10.80 | 10.77 |
| PA12 | 10.10 | | 10.54 | 10.55 |
| PA13 | 9.91 | 9.92 | 10.42 | 10.41 |
| PE01 | 14.22 | 13.82 | 14.30 | 14.01 |
| PE02 | 14.22 | 13.43 | 14.27 | 13.92 |
| PE03 | 12.95 | 12.78 | 13.62 | 13.44 |
| PE04 | 12.71 | 12.70 | 13.61 | 13.43 |
| PE05 | 12.55 | 12.58 | 13.30 | 13.13 |
| PE06 | 12.28 | 12.24 | 13.27 | 12.98 |
| PE07 | 11.97 | | 12.76 | 12.69 |
| PE08 | 11.96 | 11.95 | 12.75 | 12.69 |
| PE11 | 12.96 | | 13.06 | |
| PE12 | 11.95 | 11.92 | 12.73 | 12.67 |
| PE13 | 11.94 | 11.91 | 12.73 | 12.67 |
| PE15 | 13.39 | 13.38 | 13.84 | 13.83 |
| PE16 | 13.27 | 13.26 | 13.77 | |
| PE17 | 13.07 | 13.03 | 13.54 | 13.52 |
| PE18 | 13.07 | 13.03 | 13.54 | 13.52 |
| PE19 | 12.65 | 12.66 | 13.51 | 13.46 |
| PE19 | 12.40 | 12.33 | 13.42 | 13.37 |
| PE20 | 12.38 | 12.29 | 13.40 | 13.35 |
| PE21 | 12.38 | 12.29 | 13.40 | 13.35 |
| PE22 | 11.95 | 11.91 | 12.73 | 12.67 |
| PE23 | 11.93 | 11.86 | 12.72 | 12.66 |
| PE25 | 12.40 | 11.79 | 13.12 | 12.67 |
| PE26 | 12.39 | 11.79 | 13.09 | 12.67 |
| PE27 | 12.31 | 11.76 | 12.87 | 12.66 |
| PE28 | 11.99 | 11.76 | 12.80 | 12.64 |
| PE29 | 11.89 | 11.75 | 12.77 | 12.63 |
| PE30 | 11.88 | 11.80 | 12.69 | 12.62 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel / Structure** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| PE31 | 11.87 | 11.74 | 12.69 | 12.62 |
| PE32 | 11.75 | 11.67 | 12.46 | 12.40 |
| PE33 | 11.56 | 11.39 | 12.32 | 12.24 |
| PE34 | 11.09 | 11.01 | 11.85 | 11.81 |
| PE35 | 11.40 | | 11.88 | 11.83 |
| PE36 | 11.08 | 11.00 | 11.85 | 11.81 |
| PE37 | 10.71 | 10.69 | 11.63 | 11.57 |
| PE38 | 10.70 | 10.67 | 11.63 | 11.56 |
| PE39 | 10.70 | 10.67 | 11.63 | 11.56 |
| PE40 | 10.67 | 10.63 | 11.62 | 11.55 |
| PE41 | 10.19 | | 10.76 | 10.75 |
| ROB01 | 10.88 | 10.84 | 11.27 | 11.16 |
| ROB02 | 10.85 | 10.53 | 11.24 | 11.12 |
| ROB03 | 10.68 | 10.44 | 11.08 | 10.81 |
| ROB04 | 10.59 | 10.49 | 10.97 | 10.78 |
| ROB05 | 10.57 | 10.37 | 10.96 | 10.75 |
| ROB06 | 10.07 | 10.11 | 10.58 | |
| ROB07 | 9.99 | 10.02 | 10.55 | 10.56 |
| ROB08 | 9.92 | 9.95 | 10.52 | 10.53 |
| ROB09 | 9.86 | 9.89 | 10.50 | |
| ROB10 | 9.88 | 9.91 | 10.56 | 10.57 |
| ROB11 | 9.86 | 9.89 | 10.52 | 10.53 |
| ROB12 | 9.85 | 9.87 | 10.49 | |
| ROB13 | 9.39 | 9.43 | 9.99 | 10.00 |
| SH1 | 9.53 | 9.54 | 10.29 | 10.28 |
| SH2 | 10.85 | 10.52 | 11.24 | 11.12 |
| SH3 | 10.60 | 10.62 | 11.33 | 11.38 |
| SH4 | 10.73 | 10.69 | 11.76 | 11.65 |
| SH6 | 11.25 | 11.20 | 12.25 | 12.23 |
| SH7 | 11.63 | 11.56 | 12.83 | 12.82 |
| SH9 | 11.85 | 11.81 | 13.08 | 13.03 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel / Structure** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| SH11 | 12.39 | 12.35 | 13.39 | 13.29 |
| SH12 | 12.50 | 12.47 | 13.51 | 13.41 |
| SH13 | 12.68 | 12.62 | 13.54 | 13.51 |
| SH15 | 13.33 | 13.24 | 14.32 | 14.05 |
| SH16 | 12.94 | 12.86 | 13.86 | 13.45 |
| SH17 | 12.59 | 12.02 | 13.85 | 13.43 |
| SH18 | 14.22 | 12.98 | 14.25 | 13.91 |
| SH19 | 12.68 | 12.69 | 13.61 | 13.42 |
| SH20 | 12.37 | 12.27 | 13.28 | 13.09 |
| SH21 | 12.23 | 12.14 | 13.25 | 12.96 |
| SH22 | 14.08 | 13.89 | 14.17 | 14.16 |
| SH23 | 13.27 | | 13.78 | |
| SH24 | 13.06 | 13.02 | 13.53 | 13.49 |
| SH25 | 12.35 | 12.23 | 13.37 | 13.33 |
| SH26 | 12.67 | 11.80 | 13.14 | 12.72 |
| SH27 | 12.09 | 11.76 | 12.81 | 12.66 |
| SH28 | 11.91 | 11.75 | 12.77 | 12.64 |
| SH29 | 11.86 | 11.73 | 12.68 | 12.62 |
| SH30 | 12.48 | 12.08 | 12.73 | 12.67 |
| SH31 | 11.67 | 11.52 | 12.31 | 12.29 |
| SH32 | 11.46 | 11.19 | 12.21 | 12.14 |
| SH34 | 10.94 | 10.81 | 11.75 | 11.71 |
| SH36 | 10.47 | 10.39 | 11.44 | 11.39 |
| SH37 | 12.08 | 11.75 | 13.14 | 13.06 |
| SH42 | 9.66 | 9.63 | 10.59 | 10.46 |
| SH43 | 9.84 | 9.78 | 10.84 | 10.54 |
| SH44 | 9.86 | 9.79 | 10.85 | 10.58 |
| SH45 | 9.95 | 9.94 | 10.91 | 10.69 |
| SH46 | 10.33 | | 11.03 | 10.93 |
| SH47 | 10.55 | 10.58 | 11.28 | 11.13 |
| SH48 | 11.35 | | 11.45 | 11.41 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel / Structure** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| SH49 | 11.98 | 11.03 | 12.05 | 11.64 |
| SH50 | 11.35 | 11.34 | 11.40 | |
| SH51 | 10.12 | 10.11 | 10.71 | 10.68 |
| SH52 | 10.66 | 10.59 | 10.97 | 10.96 |
| SH53 | 10.79 | 10.65 | 11.33 | 11.32 |
| SH54 | 10.58 | 10.54 | 11.22 | 11.20 |
| SH55 | 10.70 | 10.63 | 11.39 | 11.38 |
| SH56 | 10.73 | 10.68 | 11.41 | 11.39 |
| SH57 | 10.81 | 10.79 | 11.42 | 11.41 |
| SH58 | 9.47 | 9.23 | 10.35 | 10.18 |
| SH59 | 10.17 | 9.80 | 10.54 | 10.42 |
| SH60 | 10.65 | | 10.78 | |
| SX01 | 9.86 | 9.89 | 10.53 | |
| SX02 | 9.87 | 9.90 | 10.51 | |
| SX03 | 9.99 | 10.02 | 10.55 | 10.56 |
| SX04 | 10.07 | 10.11 | 10.58 | |
| SX05 | 10.79 | 10.77 | 11.79 | 11.68 |
| SX06 | 11.28 | 11.25 | 12.28 | 12.26 |
| SX07 | 11.03 | 10.76 | 11.39 | 11.18 |
| SX08 | 10.69 | 10.62 | 11.29 | 11.15 |
| SX09 | 10.43 | 10.46 | 11.05 | 10.98 |
| SX10 | 10.53 | | 11.05 | 10.96 |
| SX11 | 10.52 | | 10.97 | 10.84 |
| SX12 | 10.76 | | 11.10 | |
| SX13 | 10.07 | 10.08 | 10.95 | 10.77 |
| SX14 | 9.92 | | 10.42 | |
| SX15 | 10.65 | | 10.86 | |
| SX16 | 13.70 | | 14.26 | |
| SX17 | 9.92 | 9.95 | 10.53 | |
| SX18 | 9.48 | 9.26 | 10.36 | 10.25 |
| SX19 | 9.11 | | 9.56 | 9.57 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel / Structure** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| TN01 | 13.76 | | 14.28 | |
| TN02 | 13.20 | | 13.62 | 13.58 |
| TN03 | 13.58 | | 13.66 | |
| TN04 | 13.12 | | 13.59 | 13.55 |
| TN05 | 12.59 | 12.57 | 13.52 | 13.42 |
| TN06 | 12.46 | 12.44 | 13.40 | 13.30 |
| TN07 | 12.22 | | 13.10 | 13.05 |
| TN08 | 12.19 | | 13.10 | 13.05 |
| TN09 | 11.74 | 11.71 | 12.84 | 12.83 |
| TN10 | 11.33 | 11.30 | 12.29 | 12.28 |
| TN11 | 12.38 | | 12.51 | |
| TN12 | 11.39 | 11.38 | 12.29 | 12.27 |
| TN13 | 11.31 | 11.28 | 12.29 | 12.27 |
| TN14 | 11.28 | 11.25 | 12.28 | 12.26 |
| TN15 | 10.87 | 10.86 | 11.82 | 11.71 |
| TN15 | 10.93 | 10.92 | 11.84 | 11.73 |
| TN16 | 10.79 | 10.77 | 11.79 | 11.68 |
| TN17 | 10.62 | 10.64 | 11.38 | 11.44 |
| TN18 | 10.59 | 10.62 | 11.29 | 11.35 |

## Schedule 3 - Predicted Peak Flow Rates (upstream)

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| CN01 | 0.44 | | 1.16 | |
| CN02 | 0.30 | | 0.78 | |
| CN03 | 0.37 | 0.71 | 1.61 | 1.68 |
| CN04 | 0.22 | 0.70 | 1.60 | 1.61 |
| CN05 | 0.30 | | 0.78 | |
| CN06 | 0.29 | | 0.76 | 0.77 |
| CN07 | 0.49 | 0.89 | 2.19 | 2.02 |
| CN08 | 0.76 | 1.13 | 2.75 | 2.63 |
| CN09 | 0.74 | 1.05 | 2.49 | 2.30 |
| CN11 | 0.51 | | 1.32 | |
| CN12 | 2.37 | 0.29 | 1.26 | |
| CN13 | 0.93 | 1.23 | 2.77 | 2.85 |
| CN14 | 1.31 | 1.57 | 3.62 | 3.82 |
| CN15 | 1.50 | 1.75 | 4.13 | 4.37 |
| CN16 | 0.31 | | 0.81 | |
| CN17 | 0.29 | | 0.68 | 0.69 |
| CN18 | 1.58 | 1.82 | 3.99 | 4.36 |
| CN19 | 1.56 | 1.80 | 3.56 | 4.10 |
| CN20 | 0.54 | | 1.43 | |
| CN21 | 0.28 | | 0.73 | |
| CN22 | 0.21 | | 0.56 | |
| CN23 | 0.67 | 0.68 | 1.78 | |
| CN24 | 1.01 | | 2.72 | 2.71 |
| CN25 | 2.36 | 2.55 | 5.73 | 6.22 |
| CN26 | 2.62 | 2.79 | 6.45 | 6.98 |
| CN27 | 0.14 | | 0.36 | |
| CN28 | 2.40 | 2.62 | 4.63 | 5.67 |
| CN29 | 0.72 | | 1.92 | |
| CN30 | 2.62 | 2.83 | 5.19 | 6.31 |
| CN31 | 2.54 | 2.78 | 4.83 | 5.98 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| CN32 | 0.48 | | 1.27 | |
| CN33 | 2.82 | 3.08 | 5.47 | 6.72 |
| CN34 | 2.80 | 3.07 | 5.42 | 6.65 |
| CN35 | 3.04 | 3.31 | 5.92 | 7.43 |
| CN36 | 12.05 | 12.19 | 27.52 | 27.46 |
| GE01 | 0.74 | | 1.91 | |
| GE02 | 2.01 | 2.00 | 6.00 | 6.02 |
| GE03 | 1.95 | | 5.98 | 6.01 |
| GE04 | 1.94 | | 5.93 | 5.94 |
| GE05 | 0.54 | | 1.43 | |
| GE06 | 0.55 | 0.53 | 1.54 | 1.33 |
| GE07 | 2.66 | 2.65 | 8.03 | 7.66 |
| GE08 | 2.61 | 2.63 | 7.97 | 7.59 |
| GE10 | 2.58 | 2.60 | 7.90 | 7.53 |
| GE10 | 2.58 | 2.60 | 7.90 | 7.53 |
| MT01 | 0.33 | | 0.87 | |
| MT02 | 0.31 | | 0.66 | 0.68 |
| MT03 | 1.03 | 1.08 | 1.44 | 1.93 |
| MT04 | 0.21 | | 0.55 | |
| MT05 | 0.20 | | 0.49 | 0.50 |
| MT06 | 1.17 | 1.23 | 1.61 | 2.23 |
| MT07 | 1.38 | 1.56 | 1.70 | 2.83 |
| MT08 | 4.04 | 4.84 | 11.76 | 11.89 |
| PA01 | 0.70 | | 1.83 | |
| PA02 | 1.12 | | 2.92 | |
| PA03 | 1.55 | 1.60 | 4.50 | 4.32 |
| PA04 | 1.53 | 1.57 | 4.47 | 4.29 |
| PA05 | 1.48 | 1.51 | 4.30 | 4.13 |
| PA07 | 1.03 | 1.04 | 2.77 | |
| PA08 | 0.86 | 0.88 | 2.76 | 2.75 |
| PA09 | 1.08 | 1.12 | 4.26 | 4.19 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| PA10 | 2.34 | 2.39 | 7.31 | 7.13 |
| PA11 | 2.54 | 2.57 | 7.06 | 7.15 |
| PA12 | 2.46 | 2.48 | 6.73 | 6.86 |
| PE01 | 0.51 | | 1.36 | |
| PE02 | 0.46 | 0.50 | 1.35 | 1.33 |
| PE03 | 0.29 | 0.48 | 1.38 | 0.95 |
| PE04 | 0.82 | 0.93 | 2.09 | 1.86 |
| PE05 | 0.77 | 0.90 | 2.06 | 1.68 |
| PE06 | 0.73 | 0.88 | 1.79 | 1.69 |
| PE07 | 1.73 | 1.81 | 3.42 | 3.78 |
| PE08 | 1.72 | 1.80 | 3.30 | 3.71 |
| PE11 | 0.55 | | 1.43 | |
| PE12 | 0.16 | 0.24 | 1.04 | 0.87 |
| PE13 | 1.29 | 1.58 | 3.24 | 3.12 |
| PE15 | 1.87 | 1.72 | 5.01 | 5.00 |
| PE16 | 3.20 | 3.12 | 8.46 | 8.48 |
| PE17 | 0.27 | | 0.69 | |
| PE18 | 2.60 | 2.70 | 8.38 | 8.33 |
| PE19 | 2.52 | 2.62 | 8.26 | 8.21 |
| PE19 | 2.52 | 2.62 | 8.26 | 8.21 |
| PE20 | 0.25 | | 0.64 | |
| PE21 | 2.85 | 3.12 | 6.72 | 6.59 |
| PE22 | 2.89 | 3.22 | 7.12 | 6.97 |
| PE23 | 3.67 | 4.54 | 8.90 | 8.63 |
| PE25 | 3.13 | 3.14 | 8.93 | 9.01 |
| PE26 | 3.08 | 3.12 | 8.52 | 8.88 |
| PE27 | 3.49 | 3.54 | 9.01 | 9.21 |
| PE28 | 3.08 | 1.74 | 8.57 | 4.52 |
| PE29 | 3.43 | 2.14 | 9.38 | 5.51 |
| PE30 | 0.51 | | 1.35 | |
| PE31 | 6.45 | 6.26 | 15.44 | 14.48 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| PE32 | 6.38 | 6.24 | 15.43 | 14.48 |
| PE33 | 6.30 | 6.15 | 15.36 | 14.45 |
| PE34 | 6.59 | 6.37 | 16.18 | 15.26 |
| PE35 | 0.78 | | 2.00 | |
| PE36 | 6.72 | 6.53 | 16.60 | 15.69 |
| PE37 | 6.68 | 6.63 | 16.81 | 16.03 |
| PE38 | 0.95 | | 2.50 | |
| PE39 | 6.94 | 6.91 | 17.42 | 16.78 |
| PE40 | 6.92 | 6.89 | 17.40 | 16.77 |
| ROB01 | 0.69 | | 1.70 | 1.78 |
| ROB02 | 0.55 | 0.63 | 1.52 | 1.55 |
| ROB03 | 1.46 | 1.88 | 4.67 | 4.40 |
| ROB04 | 0.54 | | 1.41 | 1.42 |
| ROB05 | 2.13 | 2.67 | 6.42 | 6.26 |
| ROB06 | 1.99 | 2.37 | 5.41 | 5.53 |
| ROB07 | 2.61 | 3.07 | 7.02 | 7.43 |
| ROB08 | 2.51 | 2.92 | 6.58 | 6.74 |
| ROB09 | 2.76 | 3.15 | 7.26 | 7.44 |
| ROB10 | 2.40 | | 6.90 | 6.89 |
| ROB11 | 2.51 | 2.47 | 7.15 | 6.98 |
| ROB12 | 4.31 | 4.74 | 11.70 | 12.11 |
| TN01 | 0.71 | | 1.89 | |
| TN02 | 0.57 | | 1.81 | |
| TN03 | 0.20 | | 0.51 | |
| TN04 | 1.08 | | 2.95 | 3.00 |
| TN05 | 1.01 | 1.02 | 2.23 | 2.16 |
| TN06 | 1.59 | 1.62 | 3.11 | 3.38 |
| TN07 | 1.98 | 2.01 | 3.97 | 4.23 |
| TN08 | 1.98 | 2.01 | 3.88 | 4.14 |
| TN09 | 1.87 | 1.91 | 3.20 | 3.27 |
| TN10 | 2.92 | 3.00 | 5.38 | 5.47 |

| | 5-year ARI | | 100-year ARI | |
|---|---|---|---|---|
| **Channel** | **Existing** | **Maintained** | **Existing** | **Maintained** |
| TN11 | 0.53 | | 1.38 | |
| TN12 | 0.52 | 0.53 | 1.33 | 1.34 |
| TN13 | 3.06 | 3.15 | 5.77 | 5.84 |
| TN14 | 3.01 | 3.12 | 6.07 | 6.13 |
| TN15 | 3.34 | 3.50 | 6.90 | 7.11 |
| TN15 | 3.34 | 3.50 | 6.90 | 7.11 |
| TN16 | 3.23 | 3.40 | 6.46 | 6.82 |
| TN17 | 3.40 | 3.58 | 6.97 | 7.42 |
| TN18 | 6.54 | 6.82 | 15.57 | 16.07 |

Appendix C

# Disclaimers & Calculations

# Disclaimers

This Report for South Hedland Flood Study ("Report"):

- has been prepared by GHD Pty Ltd ("GHD") for the Town of Port Hedland;
- may only be used and relied on by Town of Port Hedland;
- must not be copied to, used by, or relied on by any person other than the Town of Port Hedland without the prior written consent of GHD;
- may only be used for the purposes described specifically in this report (and must not be used for any other purpose).

GHD and its servants, employees and officers otherwise expressly disclaim responsibility to any person other than the Town of Port Hedland arising from or in connection with this Report.

To the maximum extent permitted by law, all implied warranties and conditions in relation to the services provided by GHD and the Report are excluded unless they are expressly stated to apply in this Report.

The services undertaken by GHD in connection with preparing this Report were limited to those specifically detailed in Section 1 of this Report.

GHD has prepared this Report on the basis of information provided by the Town of Port Hedland, Landgate and AAM Surveys, which GHD has not independently verified or checked ("Unverified Information") beyond the agreed scope of work.

GHD expressly disclaims responsibility in connection with the Unverified Information, including (but not limited to) errors in, or omissions from, the Report, which were caused or contributed to by errors in, or omissions from, the Unverified Information.

The opinions, conclusions and any recommendations in this Report are based on assumptions made by GHD when undertaking services and preparing the Report ("Assumptions"), including (but not limited to):

- the recurrence of design rainfall;
- runoff catchment delineation;
- runoff coefficients; and
- energy loss coefficients (for hydraulic calculations).

GHD expressly disclaims responsibility for any error in, or omission from, this Report arising from or in connection with any of the Assumptions being incorrect.

Subject to the paragraphs in this section of the Report, the opinions, conclusions and any recommendations in this Report are based on conditions encountered and information reviewed at the time of preparation and may be relied on 12 months, after which time, GHD expressly disclaims responsibility for any error in, or omission from, this Report arising from or in connection with those opinions, conclusions and any recommendations.

GHD has prepared the preliminary cost estimates set out in section 6.4 of this Report ("Cost Estimate"):

- using information reasonably available to the GHD employee(s) who prepared this Report; and
- based on assumptions and judgments made by GHD.

The Cost Estimate has been prepared for the purpose of assessing the relative magnitude of specific works described in this report and must not be used for any other purpose.

The Cost Estimate is a preliminary estimate only. Actual prices, costs and other variables may be different to those used to prepare the Cost Estimate and may change. Unless as otherwise specified in this Report, no detailed quotation has been obtained for actions identified in this Report. GHD does not represent, warrant or guarantee that the works can or will be undertaken at a cost which is the same or less than the Cost Estimate.

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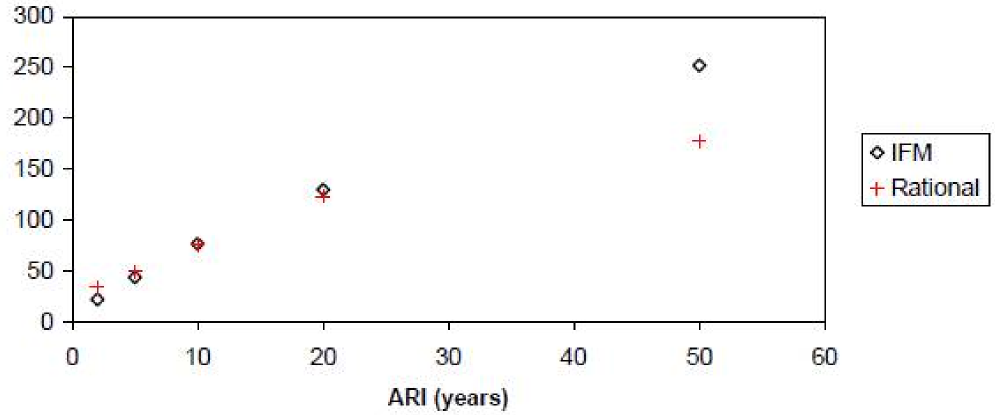
Whilst every care has been taken to prepare maps presented within this report, GHD and Landgate make no representations or warranties about their accuracy, reliability, completeness or suitability for any particular purpose and cannot accept liability and responsibility of any kind (whether in contract, tort or otherwise) for any expenses, losses, damages and/or costs (including indirect or consequential damage) which are or may be incurred by any party as a result of the map being inaccurate, incomplete or unsuitable in any way and for any reason.

## South Creek, Peak Flow Calculations

ARR 87 Regional Methods for Peak Flows in Rural Catchments

| | | | |
|---|---|---|---|
| Job No. | 6125491 | Rev No. | 0 |
| Calc. by | KN | Date | 24-Sep-10 |
| Checked by | | Date | |




**Pilbara with Loamy Soils**

Based upon data from 12 catchments with the following range of catchment characteristics

| | | |
|---|---|---|
| A | 40.5 - 7980 | km2 |
| L | 10 - 194 | km |
| Se | 1.43 - 3.77 | m/km |
| P | 230 - 400 | mm |

**Catchment Parameters**

| | | | |
|---|---|---|---|
| Area | A = | 29 | km2 |
| Length | L = | 10 | km |
| Annual Precip' | P = | 310 | mm |

**Rational Method**

It is noted that the error in the relationship is measured as

r 0.47

SSE 0.164 (+45.7%, -31.4%)

| | | | | |
|---|---|---|---|---|
| tc | =0.56 x A ^ 0.38 | = | 2.01 | hrs |
| C(2) | =3.07 x 10 ^ -1 x L ^ -0.20 | = | 0.19 | |
| I(tc) | from IFD (BoM, 2010) | = | 21.9 | mm/hr |
| Q(5) | = 0.278 x C(2) x T(tc) x A | = | 34.2 | m3/s |

| ARI | Cy/C2 | C | Q |
|---|---|---|---|
| 2 | 1 | 0.19 | 34 |
| 5 | 1.46 | 0.28 | 50 |
| 10 | 2.21 | 0.43 | 76 |
| 20 | 3.6 | 0.70 | 123 |
| 50 | 5.2 | 1.01 | 178 |

**Index Flood**

Qy/Q5

| Area | ARI 2 | 5 | 10 | 20 | 50 |
|---|---|---|---|---|---|
| 1 | 0.55 | 1.00 | 1.58 | 2.40 | 3.90 |
| 10 | 0.52 | 1.00 | 1.70 | 2.77 | 4.90 |
| 100 | 0.50 | 1.00 | 1.81 | 3.20 | 6.90 |
| 1000 | 0.48 | 1.00 | 1.94 | 2.70 | 7.90 |
| 10000 | 0.46 | 1.00 | 20.80 | 4.25 | 9.90 |

Log10 (Qy/Q5)

| Log10 (Area) | Log10 (ARI) 0.30 | 0.70 | 1.00 | 1.30 | 1.70 |
|---|---|---|---|---|---|
| 0 | -0.26 | 0.00 | 0.20 | 0.38 | 0.59 |
| 1 | -0.28 | 0.00 | 0.23 | 0.44 | 0.69 |
| 2 | -0.30 | 0.00 | 0.26 | 0.51 | 0.84 |
| 3 | -0.32 | 0.00 | 0.29 | 0.43 | 0.90 |
| 4 | -0.34 | 0.00 | 1.32 | 0.63 | 1.00 |

Q(5) = 6.73 x 10 ^ -4 x A ^ 0.72 x P ^ 1.51 = 43.94 m3/s

| ARI | Qy/Q5 | Q |
|---|---|---|
| 2 | 0.51 | 22 |
| 5 | 1.00 | 44 |
| 10 | 1.75 | 77 |
| 20 | 2.96 | 130 |
| 50 | 5.74 | 252 |

It is noted that the error in the relationship is measured as

r 0.97

SSE 0.15 (+41.4%, -29.3%)

**GHD**

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**Document Status**

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| 1 | K. Norris | G. Yandle | | G. Yandle | | 15-02-11 |