Detailed Design Report

Hanwood Stormwater Pump and Levee Design

80518062

Prepared for
Griffith City Council

24 October 2019

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Executive Summary

Introduction

The community of Hanwood NSW is located approximately 4.5 km south of the Griffith central business district. Irrigation channels controlled by Murrumbidgee Irrigation (MI) are located parallel to the northern (DC DA) and western (DC Handepot) town boundaries, connecting to Main Drain J via channel DC A. A 1.2m x 1.2m box culvert is located under Hanwood Road.

The town of Hanwood floods from Main Drain J tail water that backs up along channel DC A and inundates the town. The floodwater enters the town via the overtopping of channel DC Handepot and DC DA and inundates the western and northern parts of the town. Inundation can last for multiple days until the tail water from Main Drain J lowers.

A number of studies and investigations have been undertaken in relation to flooding and drainage within the Griffith local government area. Two reports detail the flooding behaviour and extents in Hanwood:

- Griffith Main Drain J and Mirrool Creek Flood Study (BMT WBM, 2015); and
- Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan, the FRMSP (BMT WBM, 2015).

Project Objectives

To address flooding in Hanwood, the FRMSP recommended a concept option which included construction of an earth levee along the northern and western town boundaries, parallel to channels DC DA and DC Handepot, respectively. Stormwater pumps in the north-western corner of the town were proposed to discharge town runoff over the levee, into DC DA.

Cardno (NSW/ACT) Pty Ltd was commissioned by Griffith City Council to progress the concept design presented in the FRMSP to preliminary design where multiple options are analysed and assessed. The preliminary assessment would recommend one option for final detailed design with documentation suitable for tendering and construction.

Geotechnical Investigation

Cardno undertook a geotechnical investigation to assess insitu soil properties. Based upon this investigation and review of historical geotechnical investigations it is noted that:

- Subsurface site conditions are expected to be uniform and generally non-variable.
- The site contains a number of significant above and below ground assets.
- Uncontrolled filling typically up to 0.2m deep exists along the majority of the alignment.
- Foundations not expected to pose significant issues in terms of bearing capacity, due to deep embedment and relatively stiff or better soils.
- In terms of aggressivity to concrete elements, insitu soils are considered non-aggressive to mildly-aggressive.
- In terms of soil salinity, insitu soils are considered very saline to highly saline.
- A granular pavement overlay is proposed to facilitate the raising of Hanwood Road (Kidman Way), Mallee Street and Leonard Road.

Environmental Investigations

A Preliminary Environmental Assessment of the site and its surrounding area has been undertaken by Cardno to identify potential environmental constraints that may influence the development of design options.

The Preliminary Environmental Assessment concludes that:

- As development consent for the proposed works is not required, the proposal can be assessed under Part 5 of the EP&A Act and will be determined by Griffith City Council.

- No threatened species, population, or migratory species listed under the TSC Act, FM Act and / or EPBC Act were observed within the project site. No Commonwealth Matter of National Environmental Significance are likely to be impacted by the project.
- A Part 7-permit for dredging (Section 200 of the FM Act) and the obstruction of fish passage (Section 219 of the FM Act) would not be required for any works. No other permit or approval requirements were identified.
- No heritage or other specific environmental concerns were identified during the preliminary assessment that may warrant further investigation during the design process.

A Review of Environmental Factors of the site and its surrounding area was undertaken to document the potential environmental impacts of the works and to detail the protective measures to be implemented.

The Review of Environmental Factors concludes that the proposed works are unlikely to have any significant or long term negative environmental impacts providing the appropriate mitigation measures outlined are implemented during the works.

Consultation

In order to connect the local stormwater drainage network to the proposed stormwater pump, construction boring under a number of existing utilities around the Hanwood Road/Mallee Street intersection is required. Liaison with impacted utility providers has been undertaken.

Freeboard Analysis

A site specific freeboard assessment has been undertaken to derive the required freeboard for the 1% AEP event. The calculated freeboard allowance for the proposed Hanwood levee during a 1% AEP event is 0.33m.

Options Identification and Assessment

A total of three (3) primary options have been modelled hydraulically for the 1% AEP event to assess the impacts of the proposed levee and pump. The options considered:

1. Levee along the western and northern perimeter of the township per the concept design, without the inclusion of a stormwater pump;
2. Levee along the western and northern perimeter of the township per the concept design, with the inclusion of a stormwater pump; and
3. Reduced extent of levee along the western and northern perimeter of the township, with the inclusion of a stormwater pump.

In addition to the above primary options, a number of sub-options were analysed in order to simulate various stormwater pump station duty points.

Design Flood Event

Flood mitigation measures for residential properties are typically designed to a 1% AEP storm event plus freeboard. A 1% AEP flood event has been adopted as the design flood event with a freeboard of 0.33m.

Flood Damages and Benefit-Cost Analysis

The flood damages assessment undertaken as part of the Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan (FRMSP) has further been updated and refined for the flood damages assessment of the options identified in this report.

The AAD reduction, or the “benefit”, has been reduced to a net present value at 4%, 7% and 11% discount rates assuming a design life of 50 years. The “cost” for each option includes the capital/construction cost of the options and the annual maintenance cost over the design life of the levee.

The flood damages calculation does not include the cost of intangible damages such as trauma, accessibility issues or ongoing difficulties during a flood event. Although, none of the options provide a benefit cost ratio higher than 1, nominated options provide flood immunity to the town of Hanwood from inundation during and up to a 1% AEP event.

Recommended Option

Based on the results of the hydraulic assessment of the options, comparison with the upstream properties finished floor levels and the Benefit-Cost analysis, Option 8 has been recommended as the preferred option and progressed to detail design stage.

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1 Introduction

In response to the 2012 flood event in the Griffith region, Griffith City Council (GCC) recently completed "Griffith Main Drain J and Mirrool Creek Flood Study" and "Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan" (FRMSP) for both Main Drain J and Mirrool Creek catchments. Council has adopted the Plan in accordance with the Manual, which included recommended flood mitigation works which are considered to be at "concept design" stage.

Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan (FRMSP) has identified that flooding in Hanwood largely occurs when Main Drain J is running at capacity. The elevated water levels in Main Drain J extend backwater influences along DC A. This, together with a hydraulic gradient to drain DC A and its contributing catchments, initiates extensive out of bank flooding, including within the township of Hanwood. Flooding may last for a few days, until the tailwater level in Main Drain J lowers to enable drainage out of Hanwood.

The flows draining through Hanwood are relatively small due to the size and flat nature of the upstream catchment, which is drained via DC DA and DC Handepot. It is principally the backwater influence of flooding from Main Drain J that causes flooding within Hanwood, rather than a lack of capacity within the drainage channels to convey the local catchment runoff.

The FRMSP has recommended that the extent of the backwater flooding into Hanwood can be limited through the construction of a bund and one way valves with respect to the local flooding and drainage. The concept design for the Hanwood flooding mitigation includes:

> Approximately 700m of earthen bund constructed along left bank alignment of DC DA and DC Handepot. The bund crest is at a nominal height of 122.1m AHD (typical height of 0.7m) providing for a 1% AEP flood immunity with freeboard of approximately 0.25m;

> Provision of one-way flow structures on DC 0491D (eastern side of Hanwood Road) and DC Handepot (and any other drainage connections that might be present) to prevent elevated water levels in DC DA flowing into the area behind the bund; and

> The installation of pumps on DC 0491D and DC Handepot to discharge local catchment runoff from behind the bund into DC DA during periods when the one-way flow structures are 'locked'.

Griffith City Council has engaged Cardno to progress the concept design presented in the "Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan" (BMT WBM, 2015) to preliminary design where multiple options are analysed and assessed and ultimately, prepare a final detailed design with documentation suitable for tendering and construction.

This project is to be undertaken in two stages:

Stage 1 – Investigation and Preliminary Design

Stage 2 – Detailed Design and Documentation

The contents of this report describe the methodology, results and conclusions pertaining to **Stage 1 and Stage 2**.

2 Background

2.1 Site Location

The community of Hanwood NSW is located approximately 4.5 km south of the Griffith central business district. Main Drain J, running in an east west direction, is located approximately 1.2km north-east of Hanwood. Town drainage is connected to Main Drain J via the channels DC DA and DC Handepot, running along the northern and western town boundaries respectively, and DC A which extends between DC DA and Main Drain J. These channels are owned, operated and maintained by Murrumbidgee Irrigation (MI). A 1.2m x 1.2m box culvert crosses under Hanwood Road in channel DC DA.

Figure 2-1 gives an overview of the site and its location.

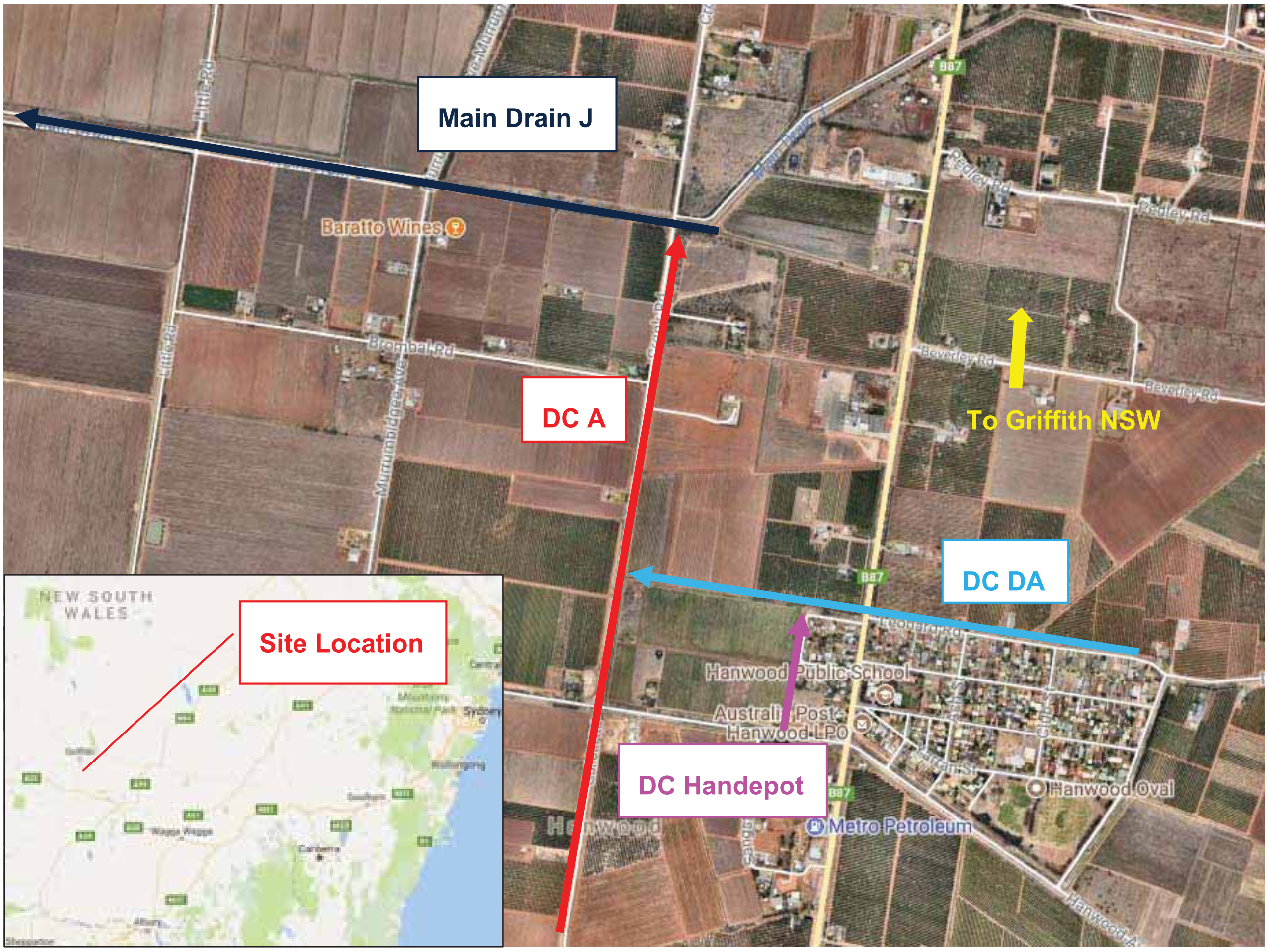


Figure 2-1 Site Location

A number of site photographs are presented below in Figure 2-2 to Figure 2-10.

Figure 2-2 DC DA parallel to Mallee Street (Looking East)

Figure 2-3 Culvert Under Hanwood Way (Looking East)

Figure 2-4 DC DA parallel to Leonard Road (Looking East)

Figure 2-5 Box Culvert Under Leonard Road (Looking North)

Figure 2-6 DC 0491D Parallel to Hanwood Road (Looking South)

Figure 2-7 DC Handepot at its confluence with DC DA (Looking South)

Figure 2-8 DN600 RCP discharging DC Handepot into DC DA (Looking North)

Figure 2-9 DC Handepot south of Wattle Street (Looking South)

Figure 2-10 Hanwood Way at intersection with Malle St (left) and Leonard Rd (right) (Looking North)

2.2 Site Inspection

Site inspections were undertaken by Cardno on 27 February 2018 and 15 May 2018. The objectives of the site inspections were to:

- Gain an appreciation of the drainage system details including the existing channels and culverts;
- Inspect the potential alignment of the proposed levee; and
- Determine the potential impacts the proposed levee may have on adjacent properties, roadways, and utilities.

The geotechnical investigation and environmental assessment inspection were undertaken on 29 April 2018.

2.3 Data Review

2.3.1 Previous Studies

A number of studies and investigations have been undertaken in relation to flooding and drainage within the Griffith local government area. Two reports address flooding in the Hanwood area:

- Griffith Main Drain J and Mirrool Creek Flood Study (BMT WBM, 2015); and
- Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan (BMT WBM, 2015).

2.3.1.1 Griffith Main Drain J and Mirrool Creek Flood Study

The Griffith Main Drain J and Mirrool Creek Flood Study (BMT WBM, 2015) (the Study) covered flooding within the Main Drain J and Mirrool Creek from just upstream of the EMR flood gates down to Barren Box Swamp and included the Hanwood residential area.

In order to accurately define flooding in the catchment, a RAFTS hydrological model and a 1D/2D TUFLOW hydrodynamic model were established and calibrated to historical flood events with assistance from the Council records and community consultation. Design flood events modelled included the 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and the probable maximum flood (PMF).

Primarily, the study seeks to understand flood behaviour in the catchment and define design flood information for use in the subsequent floodplain risk managing study and plan.

Figure 2-11 to Figure 2-13 illustrate historical flooding conditions at Hanwood in 1989 and 2012.

Figure 2-11 Flooding at Hanwood on 15 March 1989 (Source: Griffith Main Drain J and Mirrool Creek Flood Study, BMT WBM 2015)

Figure 2-12 Flooding at Hanwood on 6 March 2012 (Source: Griffith Main Drain J and Mirrool Creek Flood Study, BMT WBM 2015)

Figure 2-13 Flooding over Kidman Way on 5 March 2012 (Source: Griffith Main Drain J and Mirrool Creek Flood Study, BMT WBM 2015)

As measured at the eastern end of Beaumont Road, flood levels in channel DC A in design storm events are shown in Table 2-1. The existing road level at this location is approximately RL 122.2 mAHD

Table 2-1 Flood Levels in DC A at Hanwood

Design Flood Event Frequency	Peak Flood Level (m AHD)
20% AEP	121.2
10% AEP	121.5
5% AEP	121.7
2% AEP	121.8
1% AEP	121.9
0.5% AEP	122.0
PMF	122.2

Figure 2-14 gives an indication of the flood behaviour in Hanwood during the 1% AEP design event. Flood waters are primarily slow moving and pond within the main town area. Most of the fast flowing waters and high hazard areas are confined to the drainage channels

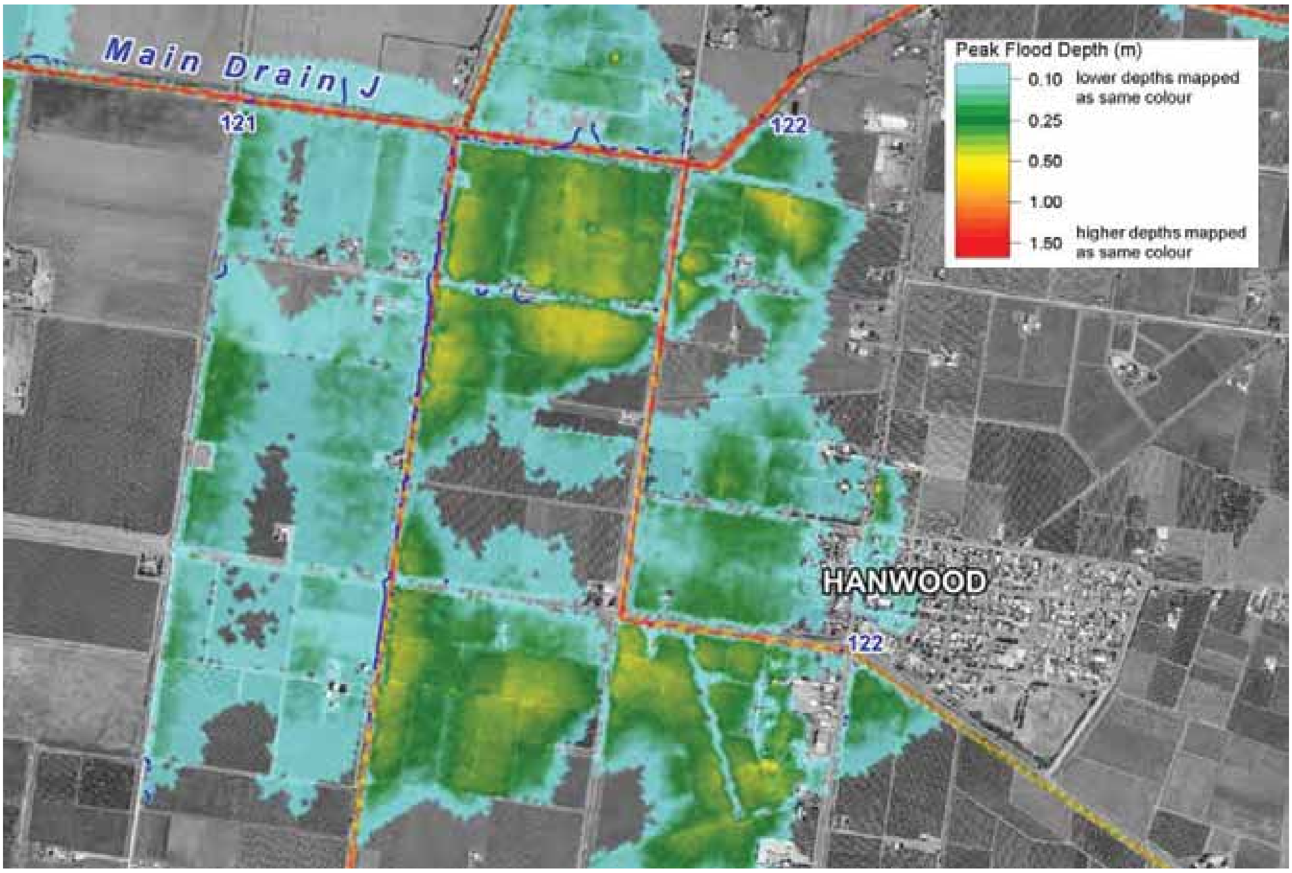


Figure 2-14 1% AEP Flood Depths at Hanwood (Source: Griffith Main Drain J and Mirrool Creek Flood Study, BMT WBM 2015)

Specific to Hanwood and the surrounding channels, the following conclusions are made in the Flood Study:

- Once catchment runoff from the flat agricultural areas in the Griffith region discharge to Main Drain J, water levels rise quickly and maintain elevated water levels for a significant time due to slow floodplain storage release.
- In Hanwood, flooding occurs when Main Drain J backwater in the fields to the west of Hanwood overtop Kidman Way.
- Suitable mitigation measures can be identified to address the existing flood risk to the established urban area in Hanwood.

2.3.1.2 Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan

Following on from the Griffith Main Drain J and Mirrool Creek Flood Study (BMT WBM, 2015), the Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan (BMT WBM, 2015) (the FRMSP) achieved the following:

- Identified and assessed measures for the mitigation of existing flood risk;
- Identified and assessed planning and development controls to reduce future flood risks; and
- Recommended floodplain management plan that outlines the best possible measures to reduce flood damages in the catchment.

To address flooding at Hanwood, the Plan recommends the construction of an earth embankment/bund along the left bank of DC DA and DC Handepot with an elevation equal to the 1% AEP flood elevation plus 250mm freeboard. In addition to a levee, the FRMSP also recommends the provision of one-way flow structures at local drainage outlets to prevent backflow into the town, and the provision of pumps to discharge local runoff over the proposed levee. This arrangement is presented in Figure 2-15.

Figure 2-15 Hanwood Flood Mitigation Measures (Source: Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan, BMT WBM 2015)

The estimated cost of the recommended flood mitigation option was $250,000 and the implementation priority was ‘Medium’.

2.3.2 Model Re-run

The 1% AEP flood event was re-run by Cardno to check for model stability and to confirm that the results produced were the same as those reported in the FRMSP.

In addition to a model re-run using TUFLOW Classic as per the original study, the 1% AEP flood event was run using TUFLOW HPC (Heavily Parallelised Compute) module. TUFLOW HPC module uses parallel computing on CPU or GPU hardware to deliver 10 to 100 times faster model runs when compared to TUFLOW Classic.

A difference map of model re-runs using TUFLOW HPC and TUFLOW Classic is presented in **Error! Reference source not found.**.

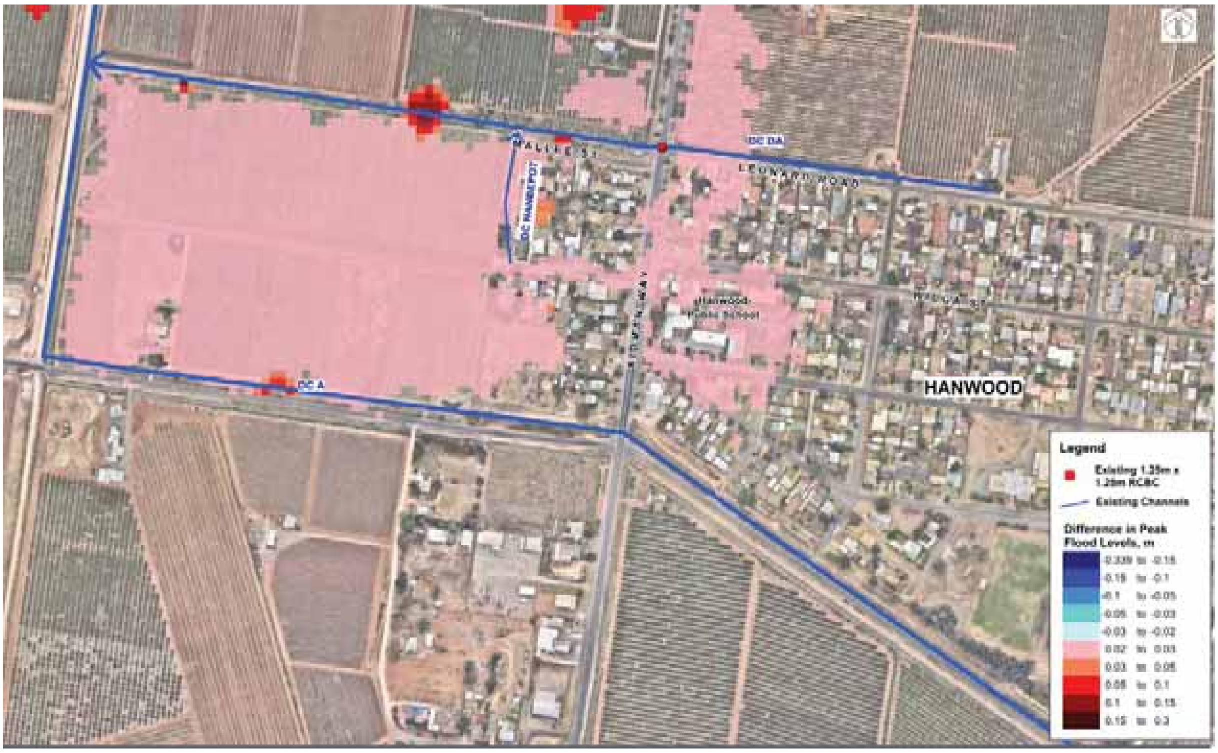


Figure 2-16 Difference Map for 1% AEP – Tuflow HPC vs Tuflow Classic

The map shows that there is a minor increase in peak flood levels (generally +0.03m) using the TUFLOW HPC module compared to the Classic module. Hence, given the minimal difference in the peak flood levels for the town of Hanwood, Cardno have utilised the TUFLOW HPC module for all further model runs for this study.

3 Geotechnical Investigation

3.1 Site Conditions

The levee is aligned along and within the northern road reserve of Leonard Road, and along the northern and western road reserve of Mallee Street. The proposed alignment intersects Hanwood Road (Kidman Way) on the northern side of parallel running Mallee Street and Leonard Rd and adjacent to DC-DA. At the time of investigation, the surrounding drainage channels, DC-DA and DC-Handepot were dry.

The overall alignment is located within regionally low-lying terrain, with local topography characterised by flat alluvial flood plains associated with the Murrumbidgee River which is located approximately 25kms to the south. Vegetation across the site comprised predominantly light grass and scattered mature trees.

A high density of underground utilities (services) were noted during the investigation in the vicinity of the Kidman Way intersection. Services included fibre optics, natural gas, telecommunications, and potable water mains which were identified by various marker posts and review of the Dial Before You Dig (DBYD) plans. Service location was undertaken by a sub-consultant in conjunction with vacuum excavation during surveying.

3.2 Subsurface Conditions

3.2.1 Levee Alignment

The subsurface profile encountered along the proposed levee alignment in boreholes (BH04-BH11) can be generally summarised as follows:

> **UNIT F - FILL**: Clayey GRAVE, Silty / Gravelly CLAY & CLAY encountered within six of the eight test bores (BH04 –BH05, BH07-BH09 & BH11) generally to depths of 0.1 m to 0.2 m BGL, with the exception of BH11 where fill was encountered up to 2.0m BGL. The materials were observed to be dry at the time of investigation, and of very stiff to hard / dense to very dense consistency. The materials are considered likely to comprise predominantly pavement materials and materials removed from the adjacent channel beds during maintenance; overlying

> **UNIT A – ALLUVIAL SOIL**: Predominantly medium to high plasticity CLAY encountered below the fill materials, with component of silt, sand and fine gravels (where present) to the depth of investigation in all locations. The material was observed to be dry of its plastic limit, and increasing in moisture content with depth increase BGL. The alluvial soils were assessed as very stiff to hard in consistency based on DCP testing undertaken.

3.2.2 Kidman Way Intersection

The subsurface conditions encountered in the three test bores drilled in the Kidman Way road shoulder pavement (BH01 – BH03) comprised:

> **WEARING COURSE**: Sprayed seal (multiple seals) to depths of 10-20 mm thickness; overlying

> **PAVEMENT**: Sandy GRAVEL pavement materials with component of silt and clay, to depths up to 0.3 m BGL. The pavement materials were observed to comprise fine to coarse, sub-rounded to angular gravels, of dry to moist condition at the time of investigation; overlying

> **SUBGRADE**: Silty / Sandy CLAY (similar to Unit A described above) at existing subgrade level to the limit of investigation. The material was observed to be dry of its plastic limit at the time of investigation, and was assessed as stiff to very stiff based on DCP testing undertaken.

Groundwater was not encountered during the investigation, however groundwater levels are likely to fluctuate with variations in climatic and site conditions. As noted the adjacent drainage channels were dry during the investigation, and groundwater may be present at the site when water is present within the channels.

Cardno's geotechnical report is included in Appendix D.

4 Environmental Assessment

4.1 Preliminary Environmental Assessment

A preliminary environmental assessment (PEA) of the site and its surrounding area has been undertaken to identify potential environmental constraints that may influence the development of design options. Once a preferred option has been chosen and a concept plan developed, a detailed environmental impact assessment (REF) would be prepared to assess the potential environmental impacts of the preferred option and would detail the environmental mitigation and management measures to be implemented.

The PEA has been prepared based on:

- Site visit;
- Review of relevant planning and legislative considerations;
- Desktop review of available Information;
- Ecological Field Survey.

Details of the findings of the preliminary assessment are detailed below:

4.1.1 Site Visit

A site visit and was undertaken by key Cardno team members on the 2nd May 2018.

4.1.2 Legislative and Planning Considerations

A preliminary review of the relevant planning and legislative context of the proposal was undertaken to determine the appropriate planning and approval pathway for the proposed works.

The study area is located within the Griffith LGA and therefore the *Griffith Local Environmental Plan 2014* applies. The works are to be located on land zoned RU5 Village and R5 under the Griffith LEP 2014.

The provisions of the State Environmental Planning Policy Infrastructure 2007 (ISEPP) as described below overrides any development consent requirements of the Griffith LEP.

ISEPP aims to facilitate the effective delivery of infrastructure across the State. Clause 50 of ISEPP permits development on any land for the purpose of flood mitigation work by or on behalf of a public authority without consent on any land.

As the chosen option would be for flood mitigation work carried out by Council, it may be assessed under Part 5 of the *Environmental Planning and Assessment Act 1979* (EP&A Act). Development consent would not be required. A Review of environmental Factors (REF) would therefore be the appropriate level of assessment required for the Proposal.

The REF would describe the proposal, document the potential impacts, and detail management and mitigation measures to be implemented to protect the environment. This assessment would be prepared pursuant to Section 111 of the *Environmental Planning and Assessment Act* (EP&A Act) 1979 and Clause 228 of the *Environmental Planning & Assessment Regulation* (EP&A Reg.) 2000.

Other relevant legislation to be considered as part of the assessment includes the:

- *Biodiversity Conservation (BC) Act 2016;*
- *Fisheries Management (FM) Act 1994;*
- *National Parks and Wildlife Act 1974;*
- *Heritage Act 1977; and the*
- *(Commonwealth) Environment Protection and Biodiversity Conservation (EPBC) Act 1999.*

No permits or approval requirements under relevant legislation were identified.

4.1.3 Desktop Review of Available Information

4.1.3.1 Flora and Fauna

A database search was undertaken using the BioNet and Protected Matters Search Tool to identify potential occurring threatened species and ecological communities within 10 km of the Study Area. The local vegetation mapping was also reviewed.

The results from the BioNet Atlas database searches indicated that 27 threatened species have been recorded within 10 km of the Study Area, including 25 bird, one mammal and one flora species. In addition, eight threatened ecological communities (TECs) are known, or are predicted to occur, within 10 km of the Study Area.

The results of the Commonwealth EPBC Protected Matters database search indicated that 18 threatened species and four TECs are known, or have potential, to occur within 10 km of the Study Area, including eight bird, three fish, one frog, two mammal and four flora species. In addition, 10 migratory species are known, or are predicted, to occur within 10 km of the Study Area.

4.1.3.2 Heritage

A search of the available online databases including the Aboriginal Heritage Information Management System (AHIMS), Australian Heritage Database (managed by the Department of Environment and Energy) and the State Heritage Register (managed by OEH) was undertaken to identify any potential Aboriginal or European heritage constraints for the project.

No Aboriginal sites or items have been listed at or near to the site. The potential to encounter previously unknown items of Aboriginal significance remains, however given the previously disturbed nature of the works location, it is considered unlikely that any sites or items of Aboriginal Significance would be disturbed by the construction or operation of the proposed works.

Listed European heritage sites located within the study area were identified on 4th June 2018 via the following sources of information:

> Australian Heritage Database;
> NSW State Heritage Register; and
> Griffith LEP (2014).

The following five (5) sites are listed as having local heritage significance on either the LEP or heritage register within Hanwood:

> Bagtown Cemetry, 731 Pedley Road, Hanwood
> Hanwood Village Store, 7 Hanwood Road (2 Yarran Street)
> Old cheese factory, Kendall Lane
> Doradillo Vine, Farm 217 Murray Road
> Griffith Centre for Irrigated Agriculture, Farm 217 Murray Road

The above mentioned items are outside the works area and will not be impacted by the proposed works. No Commonwealth or State listed heritage items or places were found to be recorded in a location that would be impacted by the construction of the proposed works.

4.1.3.3 Contamination

A search of the EPA Contaminated Land Register was undertaken on 9th April 2018. The register did not indicate any contaminated lands within the vicinity of the study area.

4.1.4 Ecological Field Survey

A field survey was undertaken on 02 May 2018. No threatened species, population or ecological communities listed under either the BC Act or EPBC Act were detected within the Study Area.

The vegetation within the Study Area was predominantly disturbed/planted. The vegetation was dominantly by introduced grass species, planted trees and regrowth. In particular, there were several planted trees along the Leonard Road and Mallee Street including *Ulmus parvifolia* (Chinese elm), *Phoenix canariensis* (Phoenix Palm), *Melaleuca linariifolia* (Snow-in-summer), *Brachychiton populneus* (Kurrajong), *Eucalyptus* sp., *Corymbia ficifolia* (Red Flowering Gum), *Callistemon* sp. (Bottlebrush) and *Acacia mearnsii* (Black Wattle).

The understorey was predominantly mowed lawns with many weed species including *Avena sativa* (Wild Oats), *Foeniculum vulgare* (Fennel), *Cirsium vulgare* (Spear Thistle), *Pennisetum clandestinum* (Kikuyu Grass), *Paspalum dilatatum* (Paspalum), *Amaranthus cruentus* (Red amaranth), *Eragrostis* sp. (Love Grass), *Verbena bonariensis* (Purpletop), *Bidens pilosa* (Cobbler's Pegs), *Taraxacum officinale* (Dandelion) and *Setaria viridis* (Pigeon Grass).

In addition, there were patches of regrowth between the private properties along the western side of the Study Area and the vineyard that included *Acacia salicina* and *Eucalyptus camaldulensis* (River Red Gum).

The fauna species detected within the Study Area were restricted to common agricultural/urban bird species, including the Starling (*Sturnus vulgaris*), Blue-faced Honeyeater (*Entomyzon cyanotis*), Common Blackbird (*Turdus merula*), Crested Pigeon (*Ocyphaps lophotes*), Yellow-rumped Thornbill (*Acanthiza chrysorrhoa*), House Sparrow (*Passer domesticus*), Willie Wagtail (*Rhipidura leucophrys*), Magpie-lark (*Grallina cyanoleuca*) and Double-barred Finch (*Taeniopygia bichenovii*).

No fauna habitat features were detected within the Study Area e.g. hollow-bearing tree, cave, rocky outcrops etc. However, a bird nest was observed in a tree located at the corner of Leonard Street and Ash Street in close proximity to the existing canal. There was no evidence that the nest was active.

The project site does not occur within the indicative distribution of any threatened fish species listed under the FM Act (DPI 2016).

No threatened species, population, ecological communities or migratory species listed under the either the BC Act or EPBC Act were detected within the Study Area. The vegetation/habitat within the Study Area is highly disturbed/modified. As such, the Study Area is limited to having sub-optimal habitat for the potentially occurring threatened species. Therefore, the proposed project is unlikely to have any significant impact on the threatened species, population (or their habitat), ecological community or migratory species listed under the BC Act or EPBC Act.

As a precaution, it is recommended that pre-clearance surveys be undertaken before the removal of any vegetation. In addition, a suitably qualified ecologist should be on standby to assist with any spotter-catcher work that may be required. In particular, the old bird nest should be inspected to ensure that it is not in use before removal and to check the nest once the tree has been felled.

4.1.5 Conclusions

As development consent for the proposed works is not required, the proposal can be assessed under Part 5 of the EP&A Act and will be determined by Griffith Council.

No threatened species, population, or migratory species listed under the TSC Act, FM Act and / or EPBC Act were observed within the project site. No Commonwealth Matter of National Environmental Significance is likely to be impacted by the project.

A part 7-permit for dredging (Section 200 of the FM Act) and the obstruction of fish passage (Section 219 of the FM Act) would not be required for any works. No other permit or approval requirements were identified.

No heritage or other specific environmental concerns were identified during the preliminary assessment that may warrant further investigation during the design process.

4.2 Review of Environmental Factors (REF)

The environmental assessment and determination of the proposal has been prepared by Cardno (NSW/ACT) Pty Ltd on behalf of Griffith City Council in accordance with Part 5 of the *Environmental Planning and Assessment Act 1979* (EP&A Act) and Clause 228 of the *Environmental Planning and Assessment Regulation* (2000). For this proposal, Griffith City Council is both a public authority proponent (EP&A Act s.1.4) and the determining authority (EP&A Act s.5.1).

The purpose of this REF is to describe the proposed works, to document the potential environmental impacts of the works and to detail the protective measures to be implemented. In doing so, the REF helps fulfil the requirements of Division 5.1 of the EP&A Act that the determining authority examine and take into account to the fullest extent possible all matters affecting, or likely to affect, the environment by reason of the activity.

The environmental assessment indicates that the proposed works would not result in significant environmental impacts for the following reasons:

- Appropriate erosion and sediment controls would be implemented to ensure minimal impacts throughout the construction phase;
- No threatened species, populations or ecologically endangered communities (EECs), including those which are matters of national environmental significance (MNES) are considered to be affected by the proposed works;
- Noise, traffic and access issues would be short-term and managed with appropriate controls;
- No known items or places of heritage significance would be affected by the proposed works; and
- Waste generation would be minimised and managed through the application of conventional appropriate methods implemented by the appointed Contractor.

The proposed works are considered consistent with the statutory and non-statutory framework in NSW. It is expected the works would result in positive impacts, with the principal benefit being the reduction in peak flood levels in the town of Hanwood.

The REF concludes that the proposed works are unlikely to have any significant or long term negative environmental impacts providing the appropriate mitigation measures outlined in the attached REF are implemented during the works.

The full REF is enclosed in Appendix G.

5 Site Survey and Utility Potholing

Detailed site survey of DC DA, DC Handepot as well as Hanwood Way, Mallee Street and Leonard Road was undertaken by PHL Surveyors on the 4 and 5 April 2018.

Utility locating and potholing was undertaken by Tim Barnes Communications on 4 April 2018 and 30 April 2018, respectively. Potholed utilities were surveyed by PHL Surveyors on 30 April 2018 prior to backfilling.

A consolidated detailed site survey with potholing data is attached in Appendix C.

6 Stakeholder Consultation

In order to connect the local stormwater drainage network to the proposed pump, construction boring under a number of existing utilities around the Hanwood Road/Mallee Street intersection is required. Telstra and Jemena were contacted to discuss the requirements for under boring beneath their assets.

Jemena advised that gas mains need to be exposed to a depth of 500mm below the impacted gas main during under boring and that a Jemena representative is required on site during works.

Telstra referred to their ‘Duty of Care’ document which states that a minimum of 2.0m clearance between boring equipment and Telstra assets is required during under boring.

Preliminary liaison with pump suppliers has also been undertaken as part of the preliminary design phase. Details of proposed pumping rates and indicative pump sizes are given in Section 9.4.

7 Hydrological and Hydraulic Assessment

7.1 Freeboard Assessment

Levee freeboard is an additional height above the design flood level to allow for events and uncertainties that could result in water levels exceeding the design flood level. Freeboard is calculated from a number of specific components, each of which can be determined with some precision or reasonably estimated from past performance. Each of these components are described below.

7.1.1 Wave Action

The design wind and wave action estimation carried out in this study have been based on the Australian Wind Loading Standard - AS/NZS1170.2 (2002) - and guidelines for estimation of waves for deep water reservoirs outlined in Design Standards No. 13 Embankment Dams - USBR (2012).

Detailed wave height, wind setup and wave run up calculations were performed using deep water equations from Design Standards No. 13 Embankment Dams - USBR (2012) and equated to a wave action freeboard height of approximately 0.5m. 1% AEP flood depths around the proposed levee are in the order of 0.2m and thus shallow water equations could be considered more appropriate for estimating wave action components of the freeboard analysis. Based on a literature review and discussion with Cardno's Dr Doug Treloar (Senior Principal - Coastal Engineering), it was determined that there is no readily available published data for shallow water wave action against levees and that a value of **0.3m** was appropriate. This is based upon an empirical rule of Water Depth x 0.7 x 2. This value is considered typical for such scenarios and is closer to the concept value of 0.25m specified in the FRMSP. The adopted value also appears more consistent with site features such as channel depths and crop heights.

7.1.2 Local Water Surge

If at any location a levee alignment is oblique to the direction of flow, the water velocities and flow directions may change locally. This might result in local flood water levels to be higher than the general flood level. These changes can be difficult to predict under flood conditions, however flood modelling results can be used to assess likely surge heights.

The surge height (hs) can be determined from the formula:

$$Hs = V^2/2g$$

Where:

Hs = surge height (m)

V = local velocity (m/sec)

Based on a velocity of approximately 0.2m/s, the surge height for the 1% AEP event is negligible.

7.1.3 Levee Settlement

Settlement of earth fill embankments can be attributed to normal post construction settlement along with effects of drying, shrinkage, cracking etc. Post construction settlement of levees can be expected to be in the order of 1% of the levee height. Hence, assuming a final levee height of up to 0.5m, levee settlement for Hanwood levee is **0.005m**.

7.1.4 Defects in Levee

Defects in levee can result from erosion, cracking, holes due to burrowing animals and dispersion cavities. These can be mitigated by regular maintenance. A defect allowance of **0.1m** is considered appropriate for the proposed Hanwood levee.

7.1.5 Joint Probability Analysis

Joint Probability Analysis is used to take account of the dependence between input variables, as well as the distribution and extremes of the individual variables.

In general, and unless there is readily available alternate data, probability classification guidelines as per Table 7-1 can be adopted. These would be combined with the freeboard component factors as relevant to determine the design freeboard for the Yoogali levee.

Table 7-1 Probability Classifications

Description	Probability
Virtually certain	0.999
Very likely	0.99
Likely	0.9
Neutral	0.5
Unlikely	0.1
Very Unlikely	0.01
Virtually Impossible	0.001

7.1.6 Design Freeboard Allowance

The calculated freeboard allowance for the proposed Hanwood levee during a 1% AEP event is **0.33m** as presented in Table 7-2 below.

Table 7-2 Hanwood Levee Freeboard Allowance for a 1% AEP Event

Freeboard Parameter	Allowance (m)	Probability	Joint Probability Component (m)
Wave Action	0.3	-	0.3
Local Water Surge	0.00	1	0.00
Settlement	0.005	0.5	0.0025
Defects	0.10	0.5	0.05
Total Freeboard Allowance, 1% AEP Event			**0.33**

7.2 TUFLOW Model Modifications

For the purpose of this study, the base TUFLOW model developed as part of the original FRMSP (BMT WBM, 2015) has been modified to incorporate some changes as below:

7.2.1 Cell Size

Due to the reduced run times of TUFLOW HPC compared to TUFLOW classic, the cell size of the model was reduced from 20m to 10m to provide a more accurate representation of the existing topography.

7.2.2 Channel DC DA and Channel DC Handepot

Channel DC DA was not included in the original modelling undertaken by BMT WBM. Hence, in order to improve the accuracy of modelled flood behaviour around Hanwood, channel DC DA has been included as a 1D network element in TUFLOW based on cross sections and levels obtained from detailed site survey undertaken by PHL Surveyors. Channel DC Handepot has been included as a topographic breakline which locally lowers the ground levels to represent the channel invert.

7.2.3 Local Hanwood Catchment

The 47ha local Hanwood catchment that drains to the proposed pumping station was removed from catchment ‘MDJ11’ the Main Drain J XP-RAFTS model. This catchment was modelled separately in XP-RAFTS with an assumed imperviousness of 60% and then applied to the TUFLOW hydraulic model as a source area boundary over the town to accurately model the local catchment runoff from the town.

7.3 Existing Scenario Flood Modelling

The updated base case TUFLOW model (as per Section 7.2) has been assessed for three design storm events.

- 1% AEP, 12 hour duration;
- 1% AEP, 20 minute duration; and

- 1% AEP, 1 hour duration.

The 12 hour duration storm is the critical duration for the existing scenario for the overall Mirrool Creek and Main Drain J catchment and produces the highest flood levels on the outside of the proposed Hanwood levee. This event was used to determine the proposed levee height based on 1% AEP flood levels with an allowance for freeboard.

The 20 minute duration storm is the critical event for the local Hanwood catchment draining to the proposed pumping station. This event was used to assess the pumping scenario where the pumping rate is equal to the peak 1% AEP inflow, with no allowance for above-ground storage.

The 1 hour duration storm is the critical event for the local Hanwood catchment draining to the proposed pumping station when a degree of above-ground storage is considered. This event was used for the purpose of assessing appropriate pumping rates based on a number of different stage-discharge relationships.

The maps of these existing design storm events are presented in Appendix A: Figure A1, Figure A8 and Figure A9.

7.4 Identification and Assessment of Primary Options

The updated flood models developed for the existing scenario form the base for the proposed scenario flood modelling.

To alleviate the existing flooding of Hanwood, various primary options were identified for further analysis. The options considered:

a. Extent of levee along DC DA and DC Handepot;

b. Retention or raising of intersection at Hanwood Road/Leonard Road/Mallee Street; and

c. Capacity of proposed stormwater pump.

As noted in Council's Brief, the two separate stormwater pump stations shown in the concept design have been consolidated into a single, larger stormwater pump station. The pump station will be located north of Mallee Street, to the west of the existing Telstra Exchange.

Table 7-3 presents a summary of primary options assessed.

Table 7-3 Summary of Primary Options Assessed

Primary Option Number	Extent of Levee Along DC DA	Extent of Levee Along DC Handepot	Raise Intersection of Handwood Road, Leonard Road and Mallee Street	Assumed Stormwater Pump Rate
1	Approximately 430m, between DC Handepot to near Ash St	Approximately 340m, between DC DA and near Beaumont Road	Yes, to match top of adjoining levee	Nil. No pump proposed
2	Approximately 430m, between DC Handepot to near Ash St	Approximately 340m, between DC DA and near Beaumont Road	Yes, to match top of adjoining levee	9cum/s (higher than storm inflow rate)
3	Approximately 170m, between DC Handepot to Hanwood Road	Approximately 160m, between DC DA and Wattle Street	No. Retain existing levels	9cum/s (higher than storm inflow rate)

7.4.2 Option 1

Option 1 includes a levee long both DC DA and DC Handepot. A stormwater pump station is not included within this option.

a. The levee along DC DA is proposed to be approximately 430m in length and will run on the left bank (southern side) of DC DA, parallel to Mallee Street and Leonard Road.

b. The levee along DC DA will likely be constructed from earth, except adjoining the Telstra Exchange on the north-western corner of the intersection of Mallee Street and Hanwood Road. At the Exchange, it may be necessary to include a small section of concrete levee to minimise the footprint of the works and thus minimise any impacts to the Exchange's infrastructure.

c. The levee along DC Handepot is proposed to be approximately 340m in length and will run for the full length of DC Handepot from near Beaumont Road to DC DA. The levee is likely to be constructed from earth and will run on the left bank (western side) of DC Handepot in private property. The proposed levee will impact the existing vehicle access and western vineyard rows. It is noted that the majority of existing DC Handepot is located on private property.

d. To maintain flood immunity to the township, it is necessary to raise Hanwood Road to match the level of the proposed levee along DC DA. Raising of Hanwood Road will require local tie-in and reconstruction of the adjoining Mallee Street and Leonard Road. Extension of the existing 1.2m W x 1.2m H RCBC under Hanwood Road and reconstruction of headwalls and vehicle guardrails will also be required along with local adjustment to utility services.

e. No stormwater pump is proposed for this option. Stormwater runoff from the township will be conveyed to the existing low point along Mallee Street and will pond.

This option isolates the township from DC DA and DC Handepot backwater impacts and intends to demonstrate the need, or otherwise, to include a stormwater pump station to manage local runoff from the township.

The preliminary design layout of Option 1 is presented on the drawings included in Appendix B.

7.4.3 Option 2

Option 2 includes a levee long both DC DA and DC Handepot. A stormwater pump station is included and will discharge to DC DA.

a. The levee along DC DA is proposed to be the same as Option 1 as described in Section 7.4.2.

b. The levee along DC Handepot is proposed to be the same as Option 1 as described in Section 7.4.2.

c. To maintain flood immunity to the township, it is necessary to raise Hanwood Road as Option 1 as described in Section 7.4.2.

d. A single stormwater pump station is proposed to be constructed to the west of the Telstra Exchange at the intersection of Hanwood Road and Mallee Street. For the purpose of this option, the pump flow rate has been assumed at 9cum/s which exceeds the peak inflow to the pump from the township catchment.

This option isolates the township from DC DA and DC Handepot backwater impacts and demonstrates township flood water behaviour prior to the local township stormwater upgrades as described in Section 9.6.

The preliminary design layout of Option 2 is presented on the drawings included in Appendix B.

7.4.4 Option 3

Option 3 includes a reduced length of levee long both DC DA and DC Handepot. A stormwater pump station is included and will discharge to DC DA.

a. The levee along DC DA is proposed to be approximately 170m in length and will run on the left bank (southern side) of DC DA, parallel to Mallee Street only, between DC Hanwood Road and DC Handepot. This reduced levee extent along DC DA will allow the exchange of stormwater flows into and out of the township and DC DA along Leonard Road. Further details are presented below in Section 7.4.4.1.

b. The levee along DC DA will likely be constructed from earth, except adjoining the Telstra Exchange on the north-western corner of the intersection of Mallee Street and Hanwood Road. At the Exchange it may be necessary to include a small section of concrete levee to minimise the footprint of the works and thus minimise any impacts to the Exchange's infrastructure.

c. The levee along DC Handepot is proposed to be approximately 160m in length and will run from Wattle Street to DC DA. The levee is likely to be constructed from earth and will run on the left bank (western side) of DC Handepot in private property. The proposed levee will impact the existing vehicle access and western vineyard rows. It is noted that the majority of existing DC Handepot is located on private property. This reduced levee extent along DC Handepot will allow the exchange of stormwater flows into and out of the township and DC Handepot at the western end of Wattle Street. Further details are presented below in Section 7.4.4.1.

d. As the levee along DC DA stops at Hanwood Road, it is not necessary to raise Hanwood Road to match the level of the proposed levee along DC DA. Associated works such as extending the existing RCBC, headwalls, guardrail and utility services will not be required.

e. A single stormwater pump station is proposed to be constructed to the west of the Telstra Exchange at the intersection of Hanwood Road and Mallee Street. For the purpose of this option, the pump flow rate has been assumed at 9cum/s which exceeds the peak inflow to the pump from the township catchment.

This option give consideration to the exchange of stormwater flows into and out of the township along Leonard Road and at the western end of Wattle Street to assess impacts on flood levels within the township. This is further discussed in Section 7.4.4.1 below.

The preliminary design layout of Option 3 is presented on the drawings included in Appendix B.

7.4.4.1 Option 3 Flow Exchange Discussion

As previously noted, it is acknowledged that the township of Hanwood is impacted by backwater from DC DA and DC Handepot. In order to further understand how and where these backwater impacts occur, a number of 2D flow lines (Plot Output (PO) lines) were added to the TUFLOW model along the township side of DC DA and DC Handepot. A total of six (6) 2D flow lines were assessed for 12 hour storm event flows into the township (i.e. backwater impacts from DC DA or DC Handepot) and flows out of the township (i.e. township flows discharging to DC DA or DC Handepot). Locations of the 2D flow lines and the velocity vectors are presented in Figure 7-1 for the existing condition.

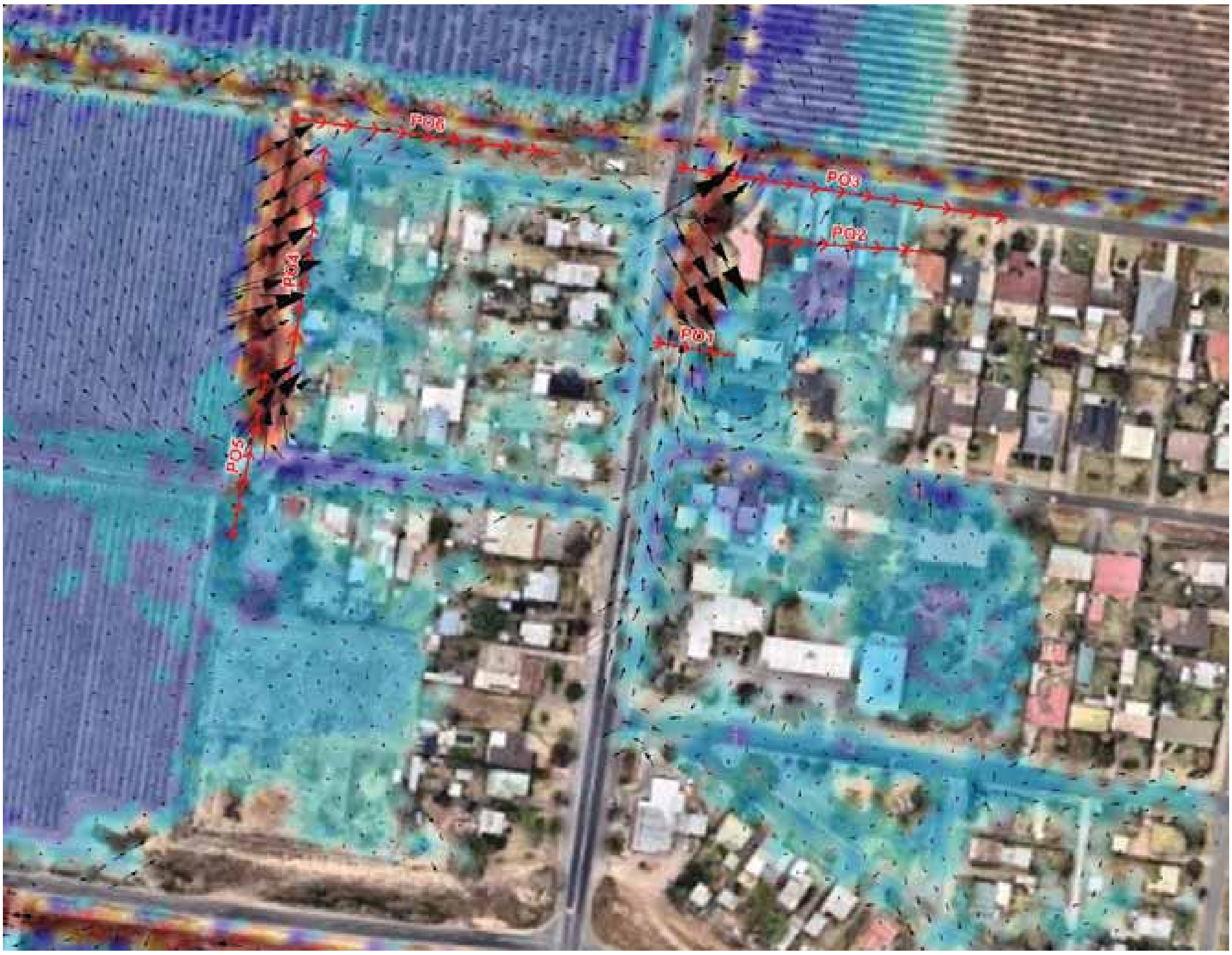


Figure 7-1 2D Flow Line Locations and Flow Vectors for the Existing 1% AEP Condition

Flows for the existing condition 1% AEP 12 hour critical storm duration were extracted for each of the 2D flow lines. A graphical representation of this data is presented below in Figure 7-2.

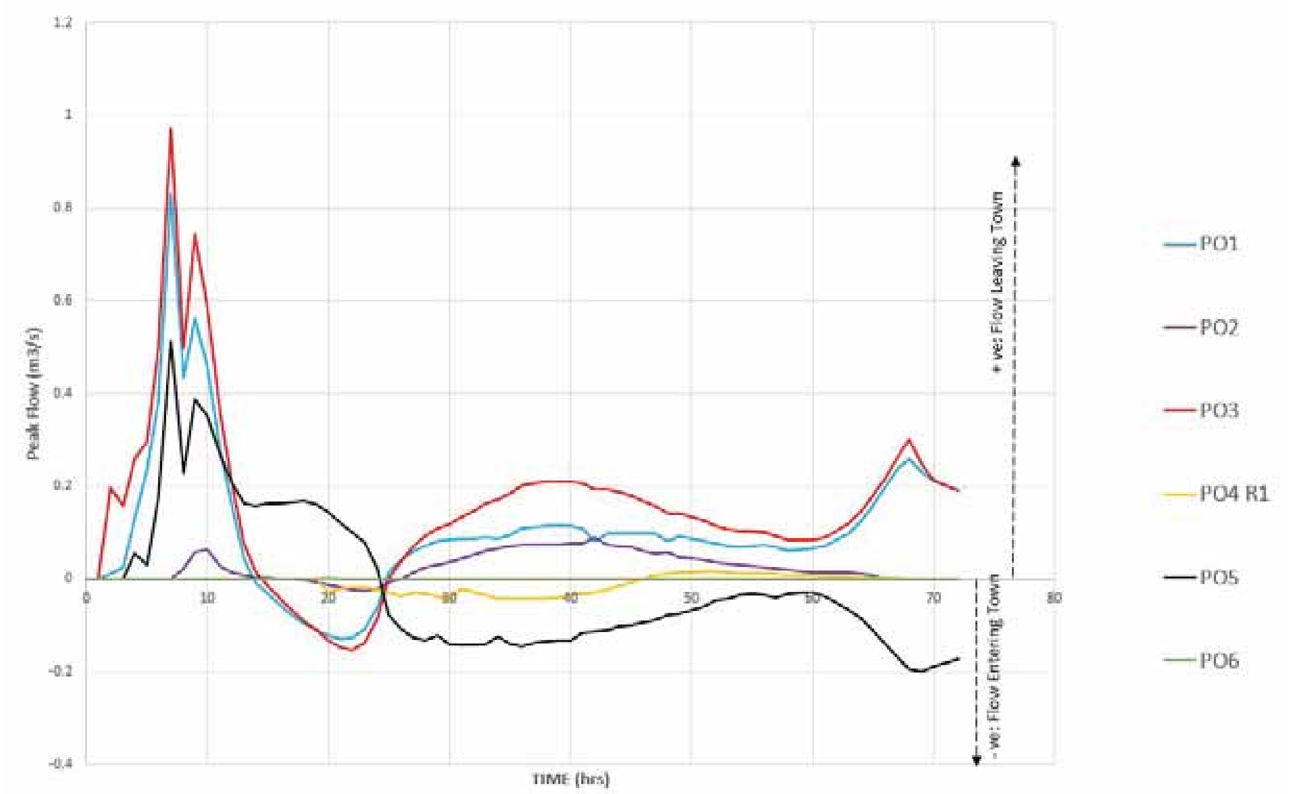


Figure 7-2 2D Flow Line Time vs Flow Plot

A summary of peak flows and flow volumes from each of the 2D flow line locations is presented in Table 7-4.

Table 7-4 Summary of 2D Flow Line Peak Flows and Flow Volumes

Description	PO1	PO2	PO3	PO4	PO5	PO6
Volume (Out of Town) (m³)	30,717	6,954	43,310	728	13,761	23
Volume (Into Town) (m³)	-3,298	-423	-3,514	-2,889	-17,704	-11
Net Volume (m³)	27,419	6,531	39,796	-2,161	-3,943	12
Peak Flow (Out of Town) (m³/s)	0.830	0.090	0.972	0.045	0.017	0.513
Peak Flow (Into Town) (m³/s)	-0.129	-0.025	-0.153	-0.017	-0.045	-0.200

Note: +ve values indicate flows leaving the town. –ve flows indicate flows entering the town.

Key Points to note from Figure 7-2 and Table 7-4 above are:

a. PO1 and PO2 flows are located inside the township, away from DC DA.

b. PO3 assesses flows into/out of the township perpendicular to Leonard Road. Results indicate that a significant volume of town stormwater enters DC DA via Leonard Road (43,300cum leaving town; 3,500cum entering town). This indicates that predominate flow behaviour at this location is water leaving the town and that there may be merits to omitting the proposed levee at this location.

c. PO4 assesses flows along DC Handepot between Wattle Street and DC DA. Results indicate that a moderate volume of stormwater enters the township from DC Handepot at this location (730cum leaving town; 2,900cum entering town). This indicates that predominate flow behaviour at this location is water entering the town, thus confirming the need for a levee at this location.

d. PO5 assesses flows at the western end of Wattle Street at DC Handepot. Results indicate that a large volume of stormwater enters and leaves the township at this location (13,700cum leaving town; 17,700cum entering town). This indicates that there is no predominate flow behaviour at this location as an approximately the same order of flows enters and leaves the town. Consideration could be given to omitting a levee at this location.

e. PO6 assesses flows into/out of the township perpendicular to Mallee Street. Results indicates that a small, insignificant volume of water is exchanged at this location (23cum leaving town; 11cum entering town). This indicates that the proposed levee is required to meet freeboard rather than to contain flows at this location.

Based upon the above assessment, Option 3 was prepared with the levee extents as described in Section 7.4.4.

7.5 Primary Options Results

Options 1, 2 and 3 as described in Section 7.4 were modelled in TUFLOW for the 1% AEP, 12 hour storm event. Flood levels were extracted at selected locations as presented in Figure 7-3.

Figure 7-3 Point Locations for Flood Levels

A summary of existing and developed condition options flood levels are presented in Table 7-5.

Table 7-5 Summary of Existing and Developed Condition Option Flood Levels

ID	Flood Level				Change in Flood Level		
	Existing	Option 1	Option 2	Option 3	Option 1 less Exist	Option 2 less Exist	Option 3 less Exist
P01	121.91	121.91	121.91	121.91	0.00	0.00	0.00
P02	121.92	121.91	121.91	121.91	-0.01	0.00	0.00
P03	121.92	121.92	121.93	121.92	0.00	0.00	0.00
P04	121.92	122.00	null	null	+0.08	Now Dry	Now Dry
P05	121.92	122.00	121.91	121.88	+0.08	-0.01	-0.04
P06	121.92	122.00	121.97	121.91	+0.09	+0.05	0.00
P07	121.92	122.00	121.87	121.91	+0.08	-0.05	-0.01
P08	121.92	122.00	121.97	121.93	+0.09	+0.05	+0.02
P09	122.05	122.05	122.05	122.05	0.00	0.00	0.00
P10	121.92	122.00	121.87	121.93	+0.08	-0.05	+0.01
P11	121.92	122.00	121.97	121.95	+0.09	+0.06	+0.04
P12	122.05	122.05	122.05	122.05	0.00	0.00	0.00
P13	122.05	122.05	122.05	122.05	0.00	0.00	0.00
P14	122.06	122.06	122.06	122.06	0.00	0.00	0.00

Note: +ve values indicate an increase flood level; -ve values indicate a decrease in flood level

A review of flood level results in Table 7-5 indicates:

7.5.2 Option 1

a. At seven (7) locations there is an increase in flood levels, typically +80mm to +90mm.

b. At one (1) location there is a decrease in flood level of -10mm (that appears to be due to rounding of flood level results)

c. At six (6) locations there is no change to existing flood levels.

d. The introduction of the levee, without the addition of a stormwater pump generally increases flood levels within the township.

e. This indicates that a stormwater pump is required to manage stormwater flows from the town.

7.5.3 Option 2

a. At three (3) locations there is an increase in flood levels, typically +50mm to +60mm.

b. The increase in flood levels at these locations appears to be attributable to the construction of the levee along DC DA parallel to Leonard Road thus preventing the discharge of town runoff to DC DA. This runoff now needs to be hydraulically conveyed west along Leonard Street to the stormwater pump station. This hydraulic conveyance along Leonard Road results in an increase in upstream flood levels at Location PO6, PO8 and PO11.

c. At four (4) locations there is a decrease in flood level of up to -50mm, including one location that is now dry (adjacent to stormwater pump).

d. The decrease in flood levels at these locations appears to be attributable to the construction of the levee along DC Handepot and the installation of the stormwater pump station.

e. At seven (7) locations there is no change to existing flood levels.

It is noted that the above results present existing, internal township local stormwater drainage conditions, without the proposed upgrades as discussed in Section 9.6.

7.5.4 Option 3

a. At three (3) locations there is an increase in flood levels, between +10mm and +40mm.

b. The increase in flood levels at these locations appears to be attributable to the reduction in levee extents and the impact of backwater from DC Handepot.

f. At three (3) locations there is a decrease in flood levels, between -10mm and -40mm, including one location that is now dry (adjacent to stormwater pump).

c. The decrease in flood levels at these locations appears to be attributable to the construction of the reduced levee extents and the installation of the stormwater pump station.

d. At eight (8) locations there is no change to existing flood levels.

e. The reduction of proposed levee extents means that the township is not isolated from backwater impacts from DC DA and DC Handepot and as such, the proposed stormwater pump station may fail to operate as expected.

7.6 Recommended Primary Option

Based upon the above results, it is suggested that Option 2 is most advantageous as it:

a. Minimises local flood level increases within the township due to the construction of the levee;

b. Maximises local flood level decreases within the township doe to the construction of the stormwater pump station; and

c. Isolates the township of Hanwood from backwater impacts from DC DA and HC Handepot.

It is noted that the above Option 2 results present existing, internal township local stormwater drainage conditions, without the proposed upgrades as discussed in Section 9.6. As noted in Section 9.6, once the future town stormwater upgrades are completed, it can be expected that significant reduction is flood levels across the township will be achieved.

8 Option 2 Levee Design

8.1 Flood Modelling

For the purpose of preliminary design, a vertical levee with an infinite height was included along the proposed alignment to determine the maximum 1% AEP flood level outside the levee. A pump with a discharge capacity equal to the peak inflow was included on the inner side of the levee to transfer local runoff over the levee into channel DC DA.

Results of the proposed model run for Option 2 for the 1% AEP, 12 hour storm event suggest the following maximum flood level on the outer side of the proposed levee at:

a. DC DA: 1% AEP flood level varies between RL121.91 to RL121.92mAHD

b. DC Handepot: 1% AEP flood level is RL121.93mAHD

The peak flood depth map for the proposed Option 2 1% AEP, 12 hour storm event is presented in Figure 8-1.

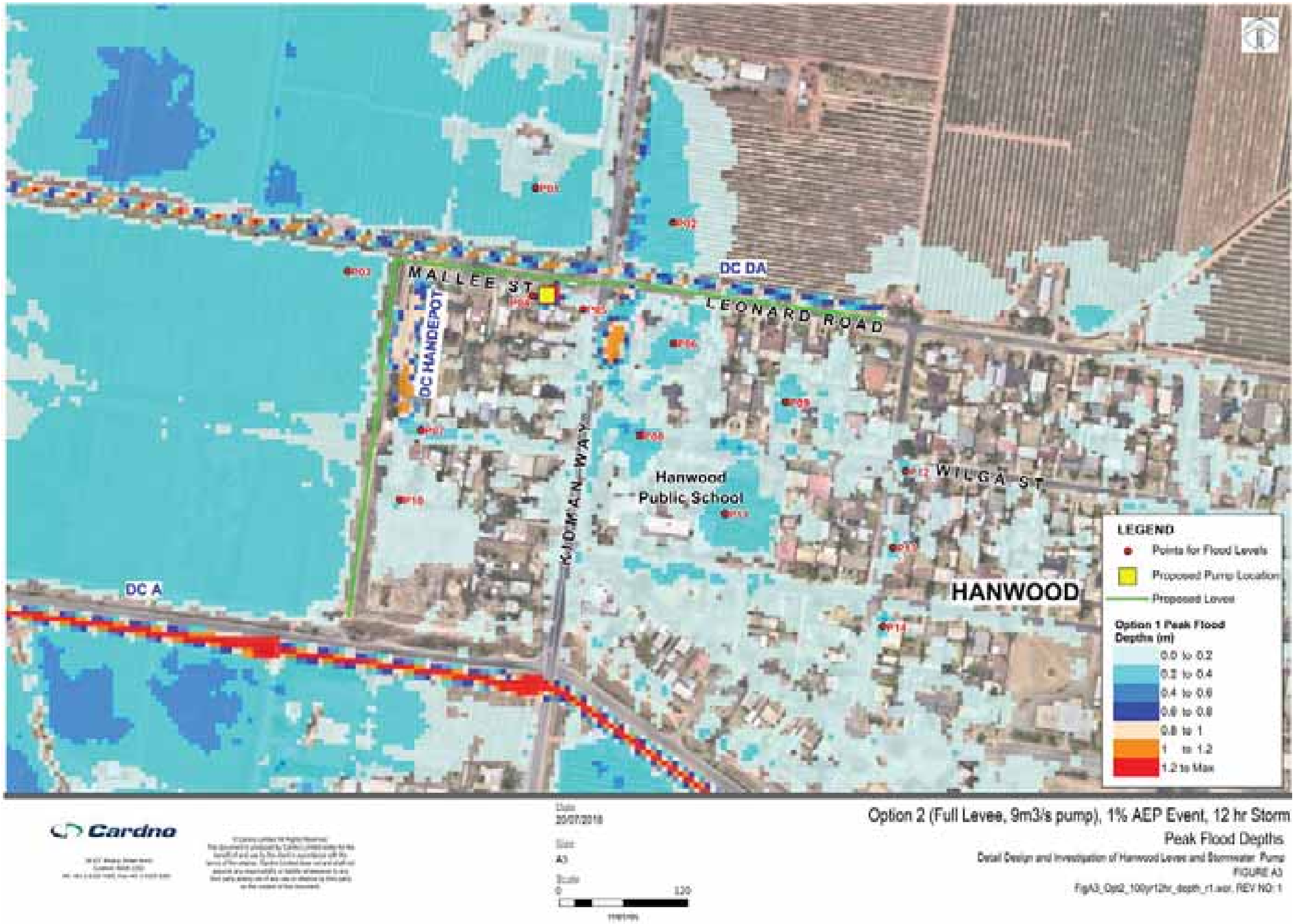


Figure 8-1 Option 2 Peak Flood Depth Map (1% AEP 12 hr storm)

8.2 Levee Alignment and Details

The table below presents a summary of minimum and adopted top of levee levels.

Table 8-1 Minimum and Adopted Top of Levee Levels

Levee Location	1% AEP Flood Level (mAHD)	Freeboard (m)	Minimum Top of Levee Level (mAHD)	Adopted Top of Levee Level (mAHD)
DC DA East	121.92	0.33	122.25	122.30
DC DA West	121.91	0.33	122.24	122.30
DC Handepot North	121.93	0.33	122.26	122.30
DC Handepot South	121.93	0.33	122.26	122.30

With consideration to the small variance between the minimum top of levee levels presented in Table 8-1 above, a uniform top of levee level of RL122.30mAHD has been adopted for the project.

It is noted that the top of levee level is determined with consideration to the flood water levels within DC DA and DC Handepot, plus freeboard. As the levee intends to protect the township from backwater flows from DC DA and DC Handepot, levee levels are not determined by flood levels within the Hanwood township.

8.3 Design Drawings

The drawings for this option have been developed to draft detailed design stage and are included in Appendix H.

9 Stormwater Pump Design

9.1 Pump Options and Storage Depth

Based upon Section 7.6 above, Option 2 is the preferred arrangement to mitigate backwater flooding from Main Drain J and DC A within the township of Hanwood. The above assessment has been conducted for the 1% AEP 12 hour storm event. This 12 hour event is the critical storm event to the wider catchment, however does not generate critical peak flows within the Hanwood township.

A preliminary assessment of proposed pumping scenarios for the local catchment storm flows has been undertaken in DRAINS and three options/scenarios have been identified with varying pump rates and allowable storage levels. The three option/scenarios are as follows:

- Scenario A (Option 4) – pumping rate to match peak inflow, nil above-ground storage.
- Scenario B (Option 5) – pumping rate less than peak inflow, above-ground storage up to typical property level.
- Scenario C (Option 6) – pumping rate less than peak inflow, above-ground storage up to -150mm below assumed dwelling FFL.

9.1.1 Scenario A (Option 4)

For Option 4, a pump flow rate was set to match the peak flow from the catchment. In this way, nil above ground storage of stormwater is required as any runoff that reaches the pump is discharged from the township side of the levee into DC DA.

This option was run purely as a theoretical 'best case' option where nil above ground storage is permitted. It is acknowledged that this option is purely theoretical and practically it cannot be achieved due to a number of constraints including pump size, electricity demand, number of starts per hour etc.

9.1.2 Scenario B (Option 5)

For Option 5, above ground stormwater storage was permitted to RL121.65mAHD. This corresponds to approximately the front boundary levels of the residential properties along the western end of Mallee Street. Resulting ponding depth is estimated at about 0.22m and permits approximately 980cum of above ground storage.

9.1.3 Scenario C (Option 6)

For Option 6, above ground stormwater storage was permitted to RL121.80mAHD. This corresponds to a level above the property boundary level along the western end of Mallee Street. This storage level is -150mm below the assumed dwelling finished floor level (FFL). Resulting ponding depth is estimated at about 0.37m and permits approximately 4,000cum of above ground storage.

For Scenario B (Option 5) and Scenario C (Option 6), above-ground stage vs. storage data was extracted from the existing 2m DEM and detailed site survey.

9.2 Pump Design AEPs and Flow Rates

While ideally the proposed stormwater pump would be designed to discharge the 1% AEP storm event, the required flow rate of the pump is likely to exceed readily available pump capacities, available electricity supply and project funding. As such, an assessment for a number of design AEPs was undertaken within each of the above options.

Table 9-1 presents a summary.

Table 9-1 Stormwater Pump Options, Design ARIs and Pump Rates

Design AEP	Pump Rate Scenario A (Option 4) (cum/s)	Pump Rate Scenario B (Option 5) (cum/s)	Pump Rate Scenario C (Option 6) (cum/s)
50%	2.4	1.2	0.2
20%	3.8	2.2	0.7
10%	4.6	2.9	1.0
1%	8.4	5.9	3.5

The above table demonstrates that Scenario B (Option 5) and Scenario C (Option 6), which include an allowance for above storage of stormwater, significantly reduce the required flow rate of the proposed pump.

Design storm hydrographs for the 50%, 20%, 10% and 1% AEP storm events were extracted from XP-RAFTS model for the local Hanwood catchment and imported into a DRAINS model to assess relationships between pumping rates and above-ground flood/storage levels for different duration events. For the purpose of preliminary design and options assessment, this approach was preferred over modelling the pump in TUFLOW due to the significantly lower run times.

Based upon the preliminary DRAINS model results, the 1% AEP local storm event was added to the TUFLOW model. These results are mapped in Appendix A: Figures A8 to A15. The TUFLOW model considers local depression storage within the catchment (using a 10m by 10m grid). These depression storages represent additional losses within the catchment and serve to further reduce the duty point of the stormwater pump (in conjunction with the allowance of above ground storage). Upgrades to the local township drainage are required to realise the full benefit of the proposed stormwater pump to ensure that the maximum volume of runoff from the catchment is conveyed to the pump and discharged to DC DA. Refer Section 9.6 for further discussion on the township stormwater upgrades.

9.3 Pump Options Results

Table 9-2 presents a summary of existing and proposed pump options for the local critical storm in Hanwood. Point ID locations are presented in Figure 7-3.

Table 9-2 Summary of 1% AEP Existing and Developed Condition Option Flood Levels

ID	Flood Level					Change in Flood Level		
	Exist 20min	Exist 60min	Option 4	Option 5	Option 6	Option 4 Less Exist	Option 5 Less Exist	Option 6 Less Exist
P01	null	null	null	null	null	No Flooding	No Flooding	No Flooding
P02	null	null	null	null	null	No Flooding	No Flooding	No Flooding
P03	null	null	121.65	121.71	121.71	Now Wet	Now Wet	Now Wet
P04	null	null	null	null	null	No Flooding	No Flooding	No Flooding
P05	null	null	null	121.82	121.82	No Flooding	Now Wet	Now Wet
P06	121.77	121.80	121.77	121.80	121.80	0.00	0.00	0.00
P07	121.58	121.66	121.54	121.68	121.68	-0.03	+0.02	+0.02
P08	121.63	121.69	121.61	121.69	121.69	-0.02	+0.01	+0.01
P09	121.97	122.01	121.92	122.01	122.01	-0.05	0.00	0.00
P10	121.79	121.81	121.79	121.81	121.81	0.00	+0.01	+0.01
P11	121.79	121.81	121.78	121.81	121.81	-0.01	0.00	0.00
P12	122.02	122.04	122.02	122.04	122.04	0.00	0.00	0.00
P13	122.02	122.04	122.00	122.04	122.04	-0.02	0.00	0.00
P14	122.02	122.05	122.02	122.05	122.05	0.00	0.00	0.00

Note: +ve values indicate an increase flood level; -ve values indicate a decrease in flood level

The above results have been prepared based upon the existing, internal township local stormwater drainage conditions, without the proposed upgrades as discussed in Section 9.6.

9.3.1 Summary of Results

A review of flood level results in Table 9-2 indicates that:

9.3.1.1 Scenario A (Option 4)

- There are no locations where there is an increase in flood levels.
- There is one (1) location that was dry that is now wet. This is where the stormwater pump station discharges near DC DA. Impacts are local and dissipate quickly to the surrounding levels.
- There are five (5) locations where there is a decrease in flood level between -10mm and -50mm.
- At eight (8) locations there is no change to existing flood levels.

It is noted that Option 4 has been modelled with a large pump with a duty that exceeds the peak stormwater discharge from the township. In this manner, all flows that reach the pump are modelled to discharge to DC DA without above ground storage. Practically, some degree of sump storage will be required to limit the number of pump starts per hour. Further details to be considered during the detailed design.

9.3.1.2 Scenario B (Option 5)

- There are three (3) locations where there is an increase in flood levels, between +10mm and +20mm.
- There are two (2) locations that were dry that are now wet. One location (PO3) is where the stormwater pump station discharges near DC DA. Impacts are local and dissipate quickly to the surrounding levels. The second location (PO5) is due to the nominated above ground storage associated with the stormwater pump station.
- There are no locations where there is a decrease in flood levels.
- At nine (9) locations there is no change to existing flood levels.

It is noted that Option 5 has been modelled with above ground storage to the property line.

9.3.1.3 Scenario C (Option 6)

- Results for Option 6 are the same as Option 5 even though there is an increase in the permitted depth of storage.

It is noted that Option 6 has been modelled with above ground storage permitted to the property line plus +150mm. A review of flood levels show that Option 6 results are the same as Option 5, where Option 5 above ground storage is permitted to the property line only.

Results indicate that the additional storage within the TUFLOW model is not required, and maximum ponding depth is not reached for Option 6. This is attributable to the hydraulic routing within the TUFLOW model and consideration of the catchment depression storage that is not considered in the DRAINS model.

9.4 Preliminary Pump Sizes

Based upon the 1% AEP design flows presented in Table 9-1, preliminary discussions were held with pump suppliers to consider options for suitable pumps. As noted in Section 9.2, it is likely that the adopted pump design ARI will be less than the 1% AEP event to manage capital costs, electricity demand and pump availability.

It is anticipated that a number of pumps will need to be included to manage stormwater flows from the township. A ‘smaller’ pump will manage flows from frequent and less server storm events while a larger single or multiple pumps may be required to manage less frequent and more sever events, up to and including the agreed design AEP event.

9.4.1 Scenario A (Option 4)

Based upon preliminary discussions with pump suppliers, twin Grundfos KPL 1400.400.12.T.50.17.L.40 50 Hz pumps have been suggested to meet the nominated 1% AEP flow rate.

Alternatively, a single KPL1800-2200 series pump could be used in lieu of the twin pumps with lower individual output capacities. As noted above, a smaller pump would be required for more frequent, less severe events in addition to the large, high-output pumps nominated above.

Refer to Appendix F for preliminary pump details.

9.4.2 Scenario B (Option 5)

Based upon preliminary discussions with pump suppliers, for a 1% AEP storm event, a single KPL1400-1800 series pump or twin KPL1200-1400 series pumps could be used.

9.4.3 Scenario C (Option 6)

Based upon preliminary discussions with pump suppliers, for a 1% AEP storm event, a single KPL 1400.400.12.T.50.17.L.40 50 Hz pump or twin KPL800-1200 series pumps could be used.

9.5 Preliminary Pump Costs

Preliminary discussion with pump suppliers has suggested an indicative per pump cost of $80,000 - $180,000 ex GST for pumps in the KPL 800-1400 range. It is noted that this price range is indicative for supply only and excludes installation, control systems and power connection.

9.6 Township Stormwater Upgrade

In order to convey flows from the township to the proposed stormwater pump station, it is necessary to undertake upgrades of the existing stormwater network within the Hanwood township. A concept design has been undertaken by Council which includes upgrades to drainage channels and town pit and pipe network.

Proposed upgrade extents span the majority of the town from the Yarran Street/Club Street intersection to the proposed pumping station to the north of Mallee Street. Due to the flat topography of the town, a typical pipe and channel grade of 0.5% has been adopted. A maximum pipe size of 750mm has been nominated in the concept design drawings; however, it is understood that no hydrological and/or hydraulic analysis has been undertaken as part of the concept upgrade design. Nominated pipe sizes may require adjustment subject to hydrological/hydraulic analysis of the town stormwater network.

As described in Section 6, connecting the town stormwater network to the proposed pump on the northern side of Mallee Street requires under boring beneath a number of existing utility services at the Hanwood Road/Mallee Street intersection. In order to provide adequate vertical clearance to Telstra assets, the invert of the proposed pipe crossing Hanwood Road will require lowering by approximately 0.4m from the levels shown on Council’s concept design. The depth of the proposed pump sump will be lowered to suit.

Council’s concept drainage upgrade drawings are included in Appendix E.

Based on the results presented in Table 9-2, it can be seen that the addition of the proposed levee without the township stormwater upgrades produces little change in flood levels compared to existing conditions. In order to improve township drainage to utilise the full benefit of the proposed stormwater upgrades mentioned previously, three stormwater upgrade options were investigated based on locations of observed ponding within the township as identified in previous modelling. Options 7, 8 and 9 comprise the addition of local stormwater networks to drain one, two and three of the observed ponding areas, respectively. The location of the properties targeted by the proposed stormwater upgrade options are indicated by the magenta (pink) linework in Figures 9-1, 9-2 and 9-3 below.

It is noted that the township stormwater upgrades have been modelled with hydraulic connections between the nominated properties and the proposed stormwater pump station. In this manner the intent and benefits of the stormwater upgrades are modelled and mapped, however the detailed design for the stormwater upgrades does not form part of the current project and will be determined as part of a future stage of the works.

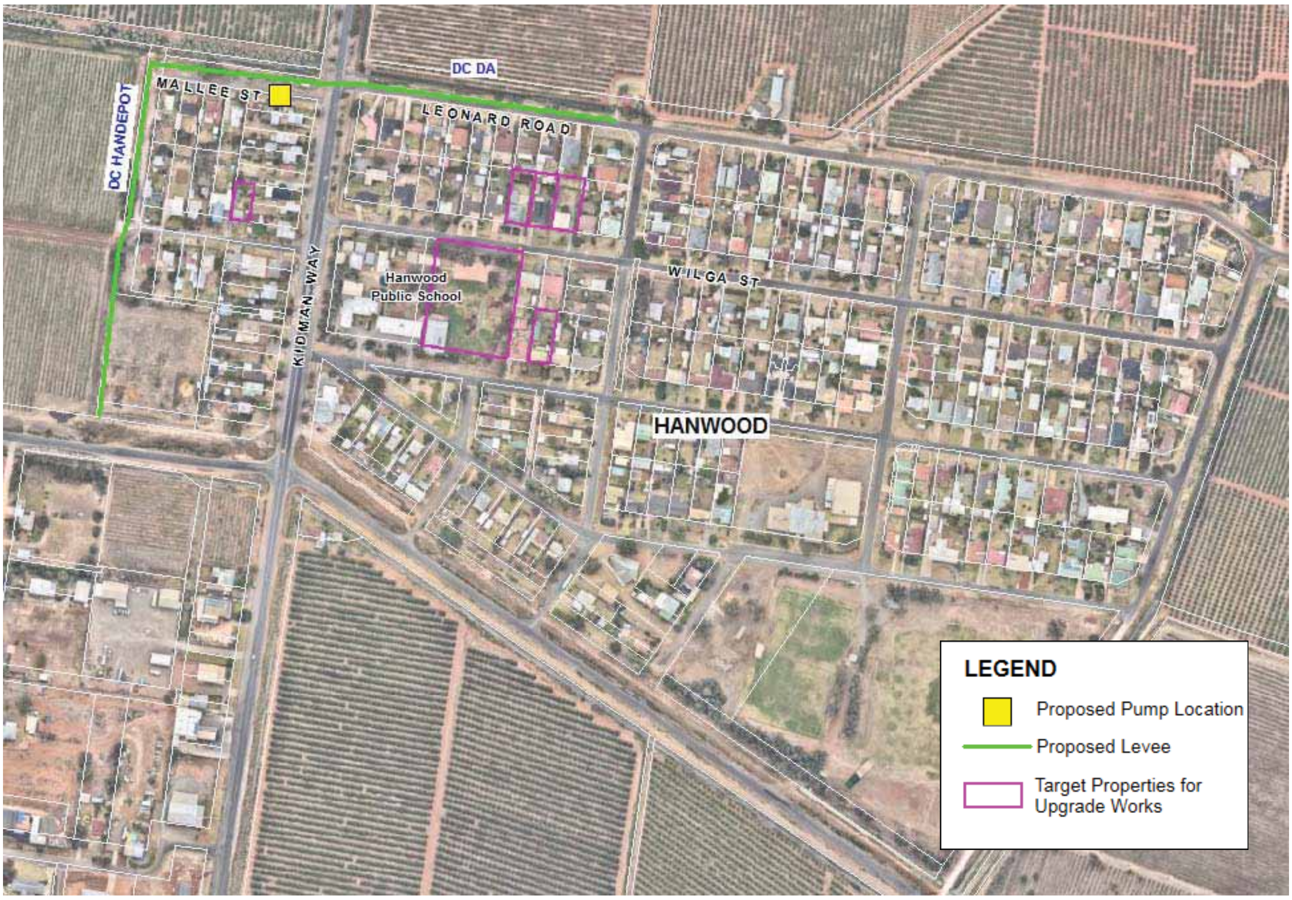


Figure 9-1 Stormwater Upgrade Option 7

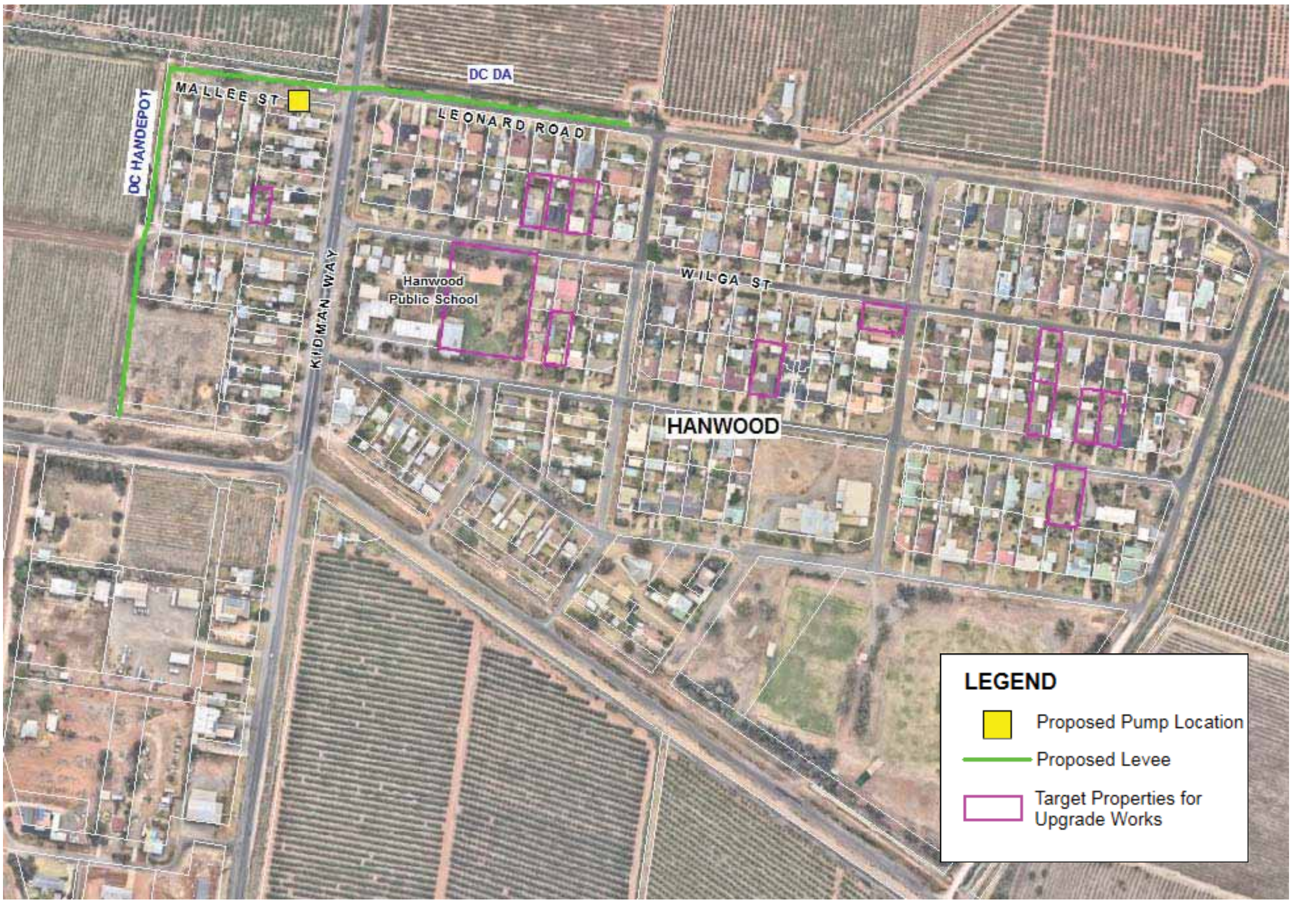


Figure 9-2 Stormwater Upgrade Option 8

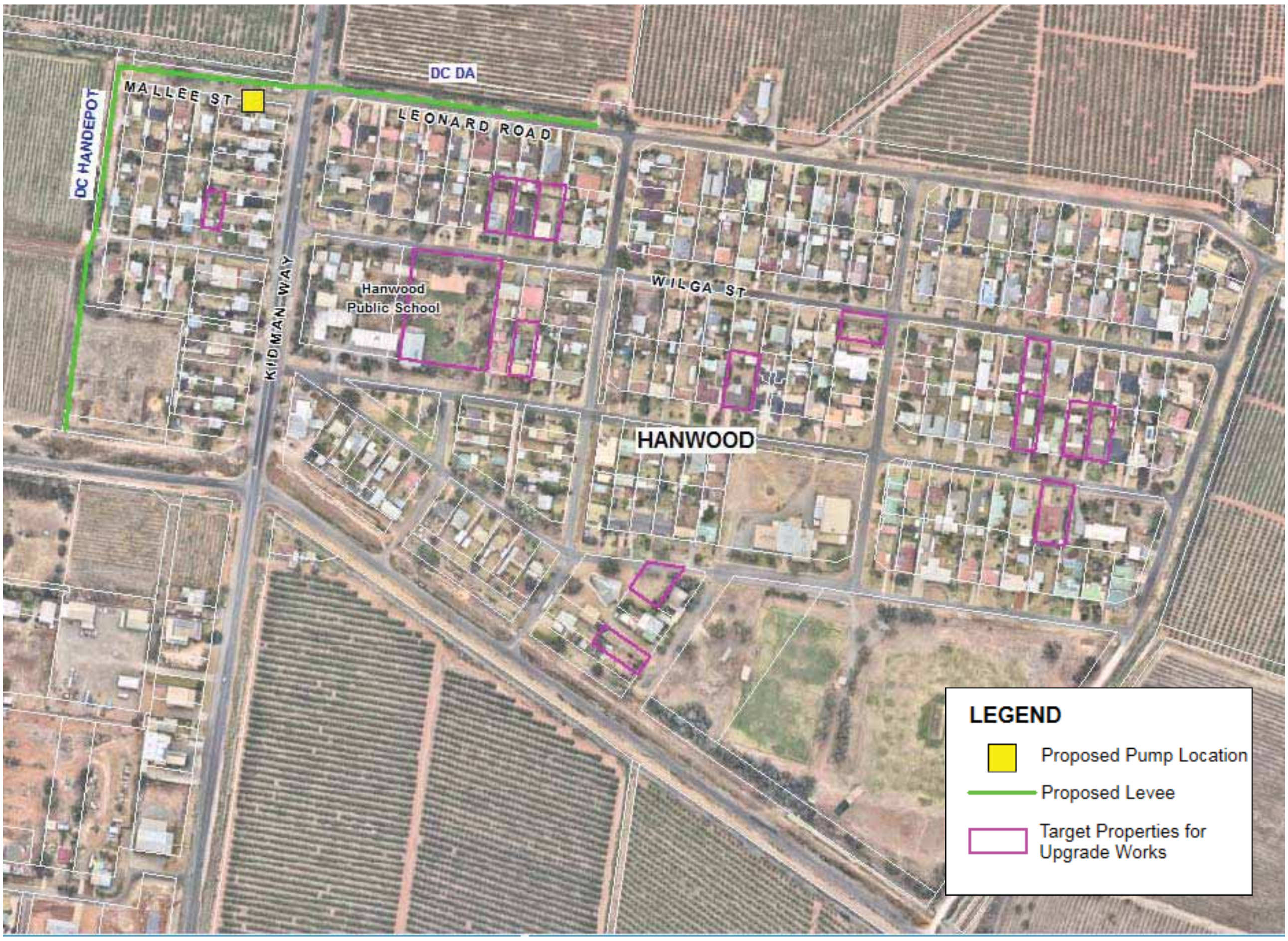


Figure 9-3 Stormwater Upgrade Option 9

Based on a sensitivity analysis, review of probable pump configurations and discussion with Council, a pumping rate of 1m^3/s has been adopted for Options 7, 8 and 9; corresponding to a 10% AEP output for pumping Scenario C.

Table 9-3 presents a summary of existing conditions and proposed pump and local drainage upgrade options for the local critical storm in Hanwood. Point ID locations are presented in Figure 7-3.

Table 9-3 Summary of 1% AEP Existing and Developed Condition Option Flood Levels

ID	Flood Level					Change in Flood Level		
	Exist 20min	Exist 60min	Option 7	Option 8	Option 9	Option 7 Less Exist	Option 8 Less Exist	Option 9 Less Exist
P01	null	null	null	null	null	No Flooding	No Flooding	No Flooding
P02	null	null	null	null	null	No Flooding	No Flooding	No Flooding
P03	null	null	121.79	121.80	121.80	Now Wet	Now Wet	Now Wet
P04	null	null	null	null	null	No Flooding	No Flooding	No Flooding
P05	null	null	121.82	121.82	121.82	Now Wet	Now Wet	Now Wet
P06	121.77	121.80	121.68	121.77	121.83	-0.09	0.00	+0.03
P07	121.58	121.66	121.35	121.72	121.81	-0.23	+0.06	+0.15
P08	121.63	121.69	121.63	121.73	121.81	0.00	+0.04	+0.12
P09	121.97	122.01	121.77	121.82	122.86	-0.2	-0.15	-0.11
P10	121.79	121.81	121.82	121.82	121.82	+0.01	+0.01	+0.01
P11	121.79	121.81	121.74	121.74	121.82	-0.05	-0.05	+0.01
P12	122.02	122.04	122.04	122.04	122.04	0.00	0.00	0.00
P13	122.02	122.04	122.04	122.04	122.04	0.00	0.00	0.00
P14	122.02	122.05	122.05	122.04	122.04	0.00	-0.01	-0.01

9.6.1 Summary of Results

A review of flood level results in Table 9-3 indicates that:

9.6.1.1 Option 7

- There is one (1) location where there is a negligible increase in increase in flood level of +10mm.
- There are two (2) locations that were dry that are now wet. One location (PO3) is where the stormwater pump station discharges near DC DA. Impacts are local and dissipate quickly to the surrounding levels. The second location (PO5) is around the pumping station where the difference between pump inflow and outflow cause minor local flooding.
- There are four (4) locations where there is a decrease in flood level between -50mm and -230mm.
- At seven (7) locations there is no change to existing flood levels.

Refer to Figure A19 in Appendix A for an illustration of the broader impacts of the Option 7 upgrade works on Hanwood flood levels.

9.6.1.2 Option 8

- There are three (3) locations where there is an increase in flood levels, between +10mm and +60mm. These locations are south of Wattle Street and at Hanwood Public School off Wilga Street. The increase in flood levels does not cause over-floor flooding.
- There are two (2) locations that were dry that are now wet. One location (PO3) is where the stormwater pump station discharges near DC DA. Impacts are local and dissipate quickly to the surrounding levels. The second location (PO5) is around the pumping station where the difference between pump inflow and outflow cause local flooding and storage at the pump. The increase in flooding is predominantly confined to the Kidman Way and Wattle Street road reserves and does not cause any notable flooding of adjacent properties.
- There are three (3) locations where there is a decrease in flood level between -10mm and -150mm. There are numerous other locations within the Hanwood Township where a decrease in flood levels is observed due to the added stormwater network.
- At six (6) locations there is no change to existing flood levels.

It is noted that the increase in flood levels around the north-western corner of Hanwood with Option 8 can be attributed to the drainage network discharging water to the pump station quicker than existing overland flow paths, resulting in a higher peak inflow at the pump station. As the pump discharge is lower than the inflow, this causes an increase in flood levels around the pump station.

Refer to Figure A20 in Appendix A for an illustration of the broader impacts of the Option 8 upgrade works on Hanwood flood levels.

9.6.1.3 Option 9

- There are five (5) locations where there is an increase in flood levels, between +10mm and +150mm.
- There are two (2) locations that were dry that are now wet. One location (PO3) is where the stormwater pump station discharges near DC DA. Impacts are local and dissipate quickly to the surrounding levels. The second location (PO5) is around the pumping station where the difference between pump inflow and outflow cause local flooding and storage at the pump. As with Option 8, the increase in flooding is predominantly confined to the road reserves; however, increases in flood levels in the order of 0.1m - 0.2m are observed along the fringe of a number of properties.
- There is one location where there is a decrease in flood level of -10mm. There are numerous other locations within the Hanwood Township where a decrease in flood levels is observed due to the added stormwater network.
- At five (5) locations there is no change to existing flood levels.

Similarly to Option 8 the increase in flood levels around the north-western corner of Hanwood can be attributed to the reduced time of concentration of the catchment and consequent increase in peak flow arriving at the pump station.

Refer to Figure A21 in Appendix A for an illustration of the broader impacts of the Option 9 upgrade works on Hanwood flood levels.

9.7 One-Way Flow Structures

To prevent backflow from DC DA into DC Handepot and DC 0491D, non-return valves will be fitted to the existing culverts at the following locations:

- DC Handepot: Fit non-return valve to existing DN600 RCP
- Leonard Road: Fit non-return valve to upgraded 2x 2.4m x 0.45m RCBCs (existing 1.2m x 0.45m RCBC)

In order to prevent possible increased local flooding due to pump failure, an emergency relief culvert fitted with an outlet non-return valve is proposed adjacent to the stormwater pump station that will extend through the levee and outlet to channel DC DA. During normal operation, this non-return value will be closed. If the pump fails during a storm event or there is a power outage, this emergency non-return valve can be manually operated, subject to levels within DC DA.

10 Benefit Cost Analysis

A flood damage assessment was undertaken as part of the Griffith Main Drain J and Mirrool Creek Floodplain Risk Management Study and Plan, which identified flood affected properties, quantified the extent of damages in economic terms for existing flood conditions and enabled the assessment of the potential flood mitigation options by means of benefit-cost analysis.

This assessment has further been updated and refined for the flood damages assessment of all the options identified in this report.

10.1 Option 2, Option 4, Option 5 and Option 6

The 1% AEP flood damages assessment for the existing scenarios, Option 2, Option 4, Option 5 and Option 6 have been shown in Tables 10-1 to 10-3 below.

Table 10-1 Summary of Damages for 1% AEP 12 hr Storm Event (Option 2)

Damage Sector	Existing Damages in Flood Event ($,000)	Option 2 Damages in Flood Event ($,000)	Reduction in Damages ($,000)
Direct Residential	591	599	-8
Indirect Residential	30	30	0
Direct Commercial	0	0	0
Indirect Commercial	0	0	0
Infrastructure and Public Sector	186	189	-3
Total	**807**	**818**	**-11**

Note: A negative (-ve) reduction in damages indicates an increase in damages

Table 10-2 Summary of Damages for 1% AEP 20min Storm Event (Option 4)

Damage Sector	Existing Damages in Flood Event ($,000)	Option 4 Damages in Flood Event ($,000)	Reduction in Damages ($,000)
Direct Residential	28	26	2
Indirect Residential	1	1	0
Direct Commercial	0	0	0
Indirect Commercial	0	0	0
Infrastructure and Public Sector	9	8	1
Total	**38**	**35**	**3**

Table 10-3 Summary of Damages for 1% AEP 60min Storm Event (Option 5 and 6)

Damage Sector	Existing Damages in Flood Event ($,000)	Option 5 Damages in Flood Event ($,000)	Option 6 Damages in Flood Event ($,000)	Existing vs Option 5 Reduction in Damages ($,000)	Existing vs Option 6 Reduction in Damages ($,000)
Direct Residential	34	34	34	0	0
Indirect Residential	2	2	2	0	0
Direct Commercial	0	0	0	0	0
Indirect Commercial	0	0	0	0	0
Infrastructure and Public Sector	11	11	11	0	0
Total	**47**	**47**	**47**	**0**	**0**

A benefit cost analysis has been undertaken to assess the total costs and relative merit of the various options, if implemented.

The estimated reduction in Annual Average Damages (AAD) for the selected options is summarised below:

- Option 2 = $-11,000
- Option 4 = $3,000
- Option 5 and 6 = $0

This change in flood damages is used as the potential saving, or benefit, to assess the economic viability of implementing the options. The "benefit", has been reduced to a net present value at 4%, 7% and 11% discount rates assuming a design life of 50 years.

The "cost" for each option includes the capital/construction cost of the options and the annual maintenance cost over the design life of the levee. For the purpose of this concept analysis, the capital/construction cost excludes any utility adjustment, protection or relocation as these works are yet to be fully scoped. It is not expected that there will be a significant difference in utility costs between the options.

A summary of the benefit-cost assessment for all the four options are presented in Table 10-4 and Table 10-5 below.

The flood damages calculation does not include the cost of intangible damages such as trauma, accessibility issues or ongoing difficulties during a flood event. Although none of the options provide a benefit cost ratio higher than 1, all the options provide flood immunity to the town of Hanwood downstream of the levee from inundation during and up to a 1% AEP event.

Table 10-4 Summary of BCR for Option 2 and Option 4

	Option 2			Option 4		
Design Life	50 years			50 years		
Annual Damages Benefit	$-11,000			$3,000		
Total Capital Construction Cost	$1,384,200			$1,384,200		
Annual Maintenance Cost	$38,200			$38,200		
NPV Discount Rates	4%	7%	11%	4%	7%	11%
Cost of Maintenance over Life of Levee	$531,215	$277,010	$133,669	$531,215	$277,010	$133,669
Total Cost over Life of Levee	$1,915,415	$1,661,210	$1,517,869	$1,915,415	$1,661,210	$1,517,869
Total Damages Benefit over Life of Levee	-$245,756	-$146,054	-$110,399	$67,024	$39,833	$30,109
Benefit Cost Ratio (BCR)	-0.13	-0.09	-0.07	0.03	0.02	0.02

Table 10-5 Summary of BCR for Option 5 and Option 6

	Option 5	Option 6
Design Life	50 years	50 years
Annual Damages Benefit	-	-
Total Capital Construction Cost	$1,276,200	$1,024,200
Annual Maintenance Cost	$33,700	$23,200

NPV Discount Rates	4%	7%	11%	4%	7%	11%
Cost of Maintenance over Life of Levee and Stormwater Pump	$468,637	$244,378	$117,923	$322,622	$168,236	$81,181
Total Cost over Life of Levee	$1,744,837	$1,520,578	$1,394,123	$1,346,822	$1,192,436	$1,105,381
Total Damages Benefit over Life of Levee	$-	$-	$-	$-	$-	$-
Benefit Cost Ratio (BCR)	0.00	0.00	0.00	0.00	0.00	0.00

The tables above indicate that:

- For Option 2, given there is an increase in flood damages, there is a negative BCR.
- Option 4 BCR ranges between 0.02 to 0.03; and
- Option 5 and Option 6 BCRs are 0 as there is no resulting benefit from the options.

Based upon the above assessment, it is noted that Option 4 has a marginally higher BCR than Option 5 and Option 6. It is further noted that Option 6 has a reduced capital and ongoing maintenance costs compared to Option 5. Based upon this, it is suggested that Option 6 is the most advantageous of the assessed options.

10.2 Option 7, Option 8 and Option 9

The BCR assessment detailed in Section 10.1 was been undertaken without consideration to the proposed township stormwater upgrades as discussed in Section 9.6 and presented in Appendix E. The township upgrades are critical to realising the full benefit of the proposed levee and stormwater pump station by improving local drainage to prevent both nuisance and backwater flooding of Hanwood properties.

The 1% AEP flood damages assessment for the existing scenarios, Option 7, Option 8 and Option 9 and have been shown in Tables 10-6, Table 10-7 and Table 10-8 below. For these options, a value of $3,500 has been assumed for yard damages. As agreed with Council, the capital and maintenance costs of the township stormwater upgrades has not been assessed in the BCR assessment below.

Table 10-6 Summary of Damages for 1% AEP 60min Storm Event (Option 7)

Damage Sector	Existing Damages in Flood Event ($,000)	Option 7 Damages in Flood Event ($,000)	Reduction in Damages ($,000)
Direct Residential	53	32	21
Indirect Residential	3	2	1
Direct Commercial	0	0	0
Indirect Commercial	0	0	0
Infrastructure and Public Sector	17	10	7
Total	**73**	**44**	**29**

Table 10-7 Summary of Damages for 1% AEP 60min Storm Event (Option 8)

Damage Sector	Existing Damages in Flood Event ($,000)	Option 8 Damages in Flood Event ($,000)	Reduction in Damages ($,000)
Direct Residential	53	11	42
Indirect Residential	3	1	2
Direct Commercial	0	0	0
Indirect Commercial	0	0	0
Infrastructure and Public Sector	17	4	13
Total	**73**	**16**	**57**

Table 10-8 Summary of Damages for 1% AEP 60min Storm Event (Option 9)

Damage Sector	Existing Damages in Flood Event ($,000)	Option 9 Damages in Flood Event ($,000)	Reduction in Damages ($,000)
Direct Residential	53	7	46
Indirect Residential	3	0	3
Direct Commercial	0	0	0
Indirect Commercial	0	0	0
Infrastructure and Public Sector	17	2	15
Total	**73**	**9**	**64**

The estimated reduction in Annual Average Damages (AAD) for the selected options is summarised below:

- Option 7 = $29,000
- Option 8 = $57,000
- Option 9 = $64,000

A summary of the benefit-cost assessment for Option 7, Option 8 and Option 9 are presented in Table 10-9, Table 10-10 and Table 10-11 below.

Table 10-9 Summary of BCR for Option 7

	Option 7		
Design Life	50 years		
Annual Damages Benefit	$29,000		
Total Capital Construction Cost	$1,024,200		
Annual Maintenance Cost	$23,200		
NPV Discount Rates	4%	7%	11%
Cost of Maintenance over Life of Levee	$322,622	$168,236	$81,181
Total Cost over Life of Levee	$1,346,822	$1,192,436	$1,105,381
Total Damages Benefit over Life of Levee	$647,903	$385,053	$291,051
Benefit Cost Ratio (BCR)	0.48	0.32	0.26

Table 10-10 Summary of BCR for Option 8

	Option 8		
Design Life	50 years		
Annual Damages Benefit	$57,000		
Total Capital Construction Cost	$1,024,200		
Annual Maintenance Cost	$23,200		
NPV Discount Rates	4%	7%	11%
Cost of Maintenance over Life of Levee	$322,622	$168,236	$81,181
Total Cost over Life of Levee	$1,346,822	$1,192,436	$1,105,381
Total Damages Benefit over Life of Levee	$1,273,464	$756,827	$572,065
Benefit Cost Ratio (BCR)	0.95	0.63	0.52

Table 10-11 Summary of BCR for Option 9

	Option 9		
Design Life	50 years		
Annual Damages Benefit	$64,000		
Total Capital Construction Cost	$1,024,200		
Annual Maintenance Cost	$23,200		
NPV Discount Rates	4%	7%	11%
Cost of Maintenance over Life of Levee	$322,622	$168,236	$81,181
Total Cost over Life of Levee	$1,346,822	$1,192,436	$1,105,381
Total Damages Benefit over Life of Levee	$1,429,854	$849,771	$642,319
Benefit Cost Ratio (BCR)	1.06	0.71	0.58

The tables above indicate that:

- Option 7 BCR ranges between 0.26 and 0.48;
- Option 8 BCR ranges between 0.52 and 0.95; and
- Option 9 BCR ranges between 0.58 and 1.06.

Based upon the above assessment, it is noted that Option 9 has a marginally higher BCR than Option 8 and a considerably higher BCR than Option 7. Due to the similar BCR of Option 8 and Option 9 and the considerable additional construction requirements for Option 9, it is recommended that Option 8 is adopted in the short term, prior to the full township drainage upgrades presented in Appendix E.

11 Detailed Design

The following sections present a discussion and elements considered during the detailed design of the Hanwood stormwater pump and levee project.

11.1 Levee Design

11.1.1 Materials

The proposed levee may be constructed from a variety of materials. The most commonly used material is earth and clay that meet predefined criteria for properties such is permeability etc. Earth levees are required to be keyed into the underlying soil. The keying and preparatory works can be difficult when utility services are present.

As an alternative, precast concrete barriers can be an economical option that can generally be installed on existing ground with minimal preparation works, thus minimising potential utility conflict.

An earth and clay levee has been adopted for the project due to its cost effective construction.

11.1.2 Cross Section Geometry

Earth levees may be constructed with a flat crest that allows vehicle access for maintenance activities. Crest width is typically 3 to 4m, depending upon the site’s spatial constraints.

Batter slopes for earth levees vary depending upon the levee height and location. Levee batter slopes may be installed at 6H:1V or flatter, up to a general maximum of 2H:1V to meet geotechnical stability requirements. It is noted that the majority of the adjoining irrigation channels have batter slopes of 2H:1V or steeper. The interaction of the irrigation channel and levee batters, along with maintenance vehicle surcharge loads, needs to be considered within the detailed design.

The use of precast concrete barriers provide a small levee footprint relative to an earth levee, but potentially require maintenance vehicle access on both sides of the barrier. Barriers should be positioned at suitable distance from the adjoining irrigation channels to minimise surcharge loadings.

A typical levee cross section with 4m top width has been adopted to allow maintenance of the adjoining channels. A 2H:1V levee batter slope has been adopted to meet stability requirements and minimise the footprint and material use of the levee. A with reduced top width levee has been adopted near the Telstra Exchange to reduce the overall levee footprint and impacts with existing site improvements.

11.1.3 Design Elevation

The top of the levee should be set a minimum of the calculated 1% AEP flood level plus design freeboard. The levee level should, as far as practical, be no higher than what it needs to be to meet design freeboard requirements to ensure that adverse impacts are not introduced to the upstream catchment during extreme storm events (i.e. greater than 1% AEP). As noted in Section 7.1.6, the design freeboard for the proposed levee is 0.33m.

With consideration to 1% AEP flood levels and design freeboard, the design elevation of the levee and intersection raising has been set to RL122.30mAHD.

11.1.4 Maintenance Access Across Levee

Access from the public road network to the Murrumbidgee Irrigation (MI) irrigation channels needs to be maintained for maintenance purposes. This will require access ramps to traverse the proposed levee.

Where an earth levee is proposed, an earth access ramp at say 10H:1V grade should be provided from the public road to the top of the levee. It is envisaged that the irrigation channel will be maintained from the crest of the levee. As the top of the levee will be offset from the top of the irrigation channel, maintenance operations may need to be undertaken with a long reach excavator.

11.2 Raised Intersections

It is proposed to raise levels of the following intersections to meet design freeboard requirements:

- Hanwood Road/Mallee Street

- Hanwood Road/Leonard Road

The intersections will be raised to form a crest at Leonard Road that matches the design level and alignment of the adjoining levee. The road adjustment requires pavement reconstruction as well as the installation of new kerb and gutter (where it currently exists), verge regrading, extension of the box culvert under Hanwood Way, and adjustment to service fitting levels and driveway accesses.

11.3 Pump Station Design

As previously noted, a pump station is proposed in the north-western corner of Hanwood to discharge local runoff over the levee and into channel DC DA. Approximate pumping rates and model sizes have been assessed as part of the design. Specific pump models, specifications and details have been nominated based upon discussions with a selection of suppliers, however, other alternatives may be acceptable, subject to approval. The pump station will include SCADA control and be connected to the local Essential Energy electricity network.

11.4 Utility Services

11.4.1 Communications

A number of underground communication assets exist along Hanwood Road, Leonard Road and Mallee Street, including a significant number of copper and optic fibre assets that service Telstra's Hanwood Exchange located on the north-west corner of the Hanwood Road and Mallee Street intersection. Telstra's Hanwood Exchange is presented in Figure 11-1.

Figure 11-1 Telstra's Hanwood Exchange on Mallee Street (Looking east)

Existing Telstra assets in Leonard Street are located under the proposed levee. While the assets will be retained, pit risers are required to suit proposed top of levee levels.

Existing Telstra assets at the intersection of Hanwood Road and Leonard Road, and Mallee Street and Hanwood Road are to be retained. Local adjustments are required to suit the proposed intersection raising.

Telstra assets within Hanwood Road have the potential to be impacted by stormwater pipe construction from DC 0491D to the proposed stormwater pump station. Telstra's Duty of Care document requires 2.0m clearance between boring equipment and Telstra assets. It is not considered economically feasible to adjust or relocate the fibre optic assets and as such, stormwater levels nominated on may need to be lowered to ensure this 2.0m clearance can be maintained.

Utility potholing indicates a minimum top of Telstra conduit level of RL 120.39 mAHD across Mallee Street. Based upon stormwater levels provided in Council's 'Proposed Drainage Upgrade for Hanwood Village' concept, there is a proposed vertical clearance of approximately 1.6m to the Telstra conduits. Invert levels have been lowered by approximately 0.4m to meet construction clearance requirements.

11.4.2 Gas

A 1050kPa secondary gas main is present on the eastern side of Hanwood Road. It is unlikely that this main will be impacted by the construction of the levee and/or road upgrades.

Utility potholing indicates a top of gas conduit level of RL 120.38 mAHD across Mallee Street and thus it is unlikely that the secondary gas main will be impacted by proposed stormwater works.

In addition to the secondary gas main, a 32mm network main is located in the southern verge of Mallee Street and a 50mm network main is located in the southern verge of Leonard Street. These local network gas mains are not expected to be impacted by the proposed works.

11.4.3 Electrical – Above Ground

A number of above ground electrical cables are present across Hanwood Road, Mallee Street and Leonard Road, in the vicinity of the proposed levee and road raising. Vertical clearance to the electrical cables is reduced due to the proposed works.

Based upon site survey, final clearance to existing above ground electrical assets have been checked and are confirmed as being greater than the minimum requirement. Verification should be undertaken on site.

11.4.4 Electrical – Below ground

Based upon information provided by Essential Energy, there are no known underground electrical assets in the area. The electrical network is above ground.

11.4.5 Potable Water

Potable water services are located around the Hanwood Road/Mallee Street intersection. It is unlikely that these services will be impacted by the construction of the levee and/or road upgrades.

Utility potholing indicates a minimum top of water conduit level of RL 120.55 mAHD across Mallee Street and thus it is unlikely that water assets will be impacted by under stormwater works.

11.4.6 Sewer

Sewer services are located outside of the subject road reserves and are not expected to be impacted by the proposed works.

12 Conclusion

This report has detailed the findings of the assessment undertaken as part of Hanwood Stormwater Pump and Levee project. The following key tasks performed:

- Review of existing studies and reports;
- Design freeboard analysis;
- Identification and assessment of flood mitigation options;
- Assessment of pumping rates;
- Recommendation of an option for detail design; and
- Preparation of the detailed design for the preferred option.

A total of nine (9) options comprising different levee extents, pumping rates and local drainage upgrades were hydraulically assessed for their suitability and resulting flood impacts. Based on the results of the hydraulic assessment of the options, comparison with the Hanwood properties finished floor levels and the Benefit-Cost analysis, Option 8 has been progressed to detail design stage.

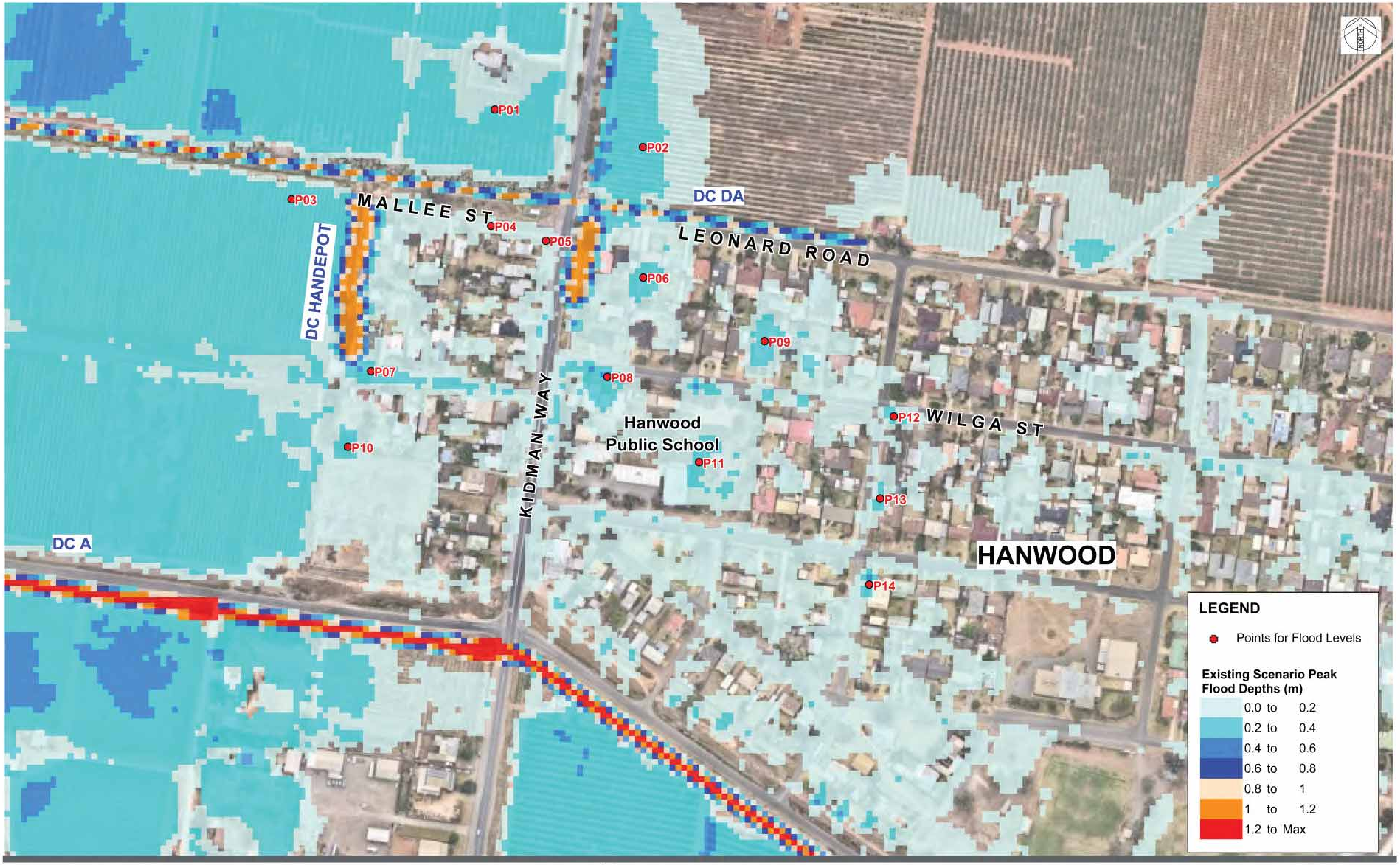
The detailed design is based upon the Option 8 layout presented in this report and has been prepared with consideration to:

- Geotechnical constraints;
- Utility service adjustment, protection or relocation;
- Clearance to overhead utility services and cover to underground utility services;
- Levee position to minimise impacts to utility services;
- Levee construction material to minimise cost and spatial footprint;
- Maintenance ramps and access to the irrigation channels;
- Configuration and grading of raised intersections; and
- Pump selection and pump station details.

APPENDIX

A

FLOOD MAPS



Existing Scenario 1% AEP Event, 12 hr Storm Duration

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A1

FigA1_Existing_100yr12hr_depth.wor, REV NO: 0

Date
13/07/2018

Size
A3

Scale
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metres

Cardno

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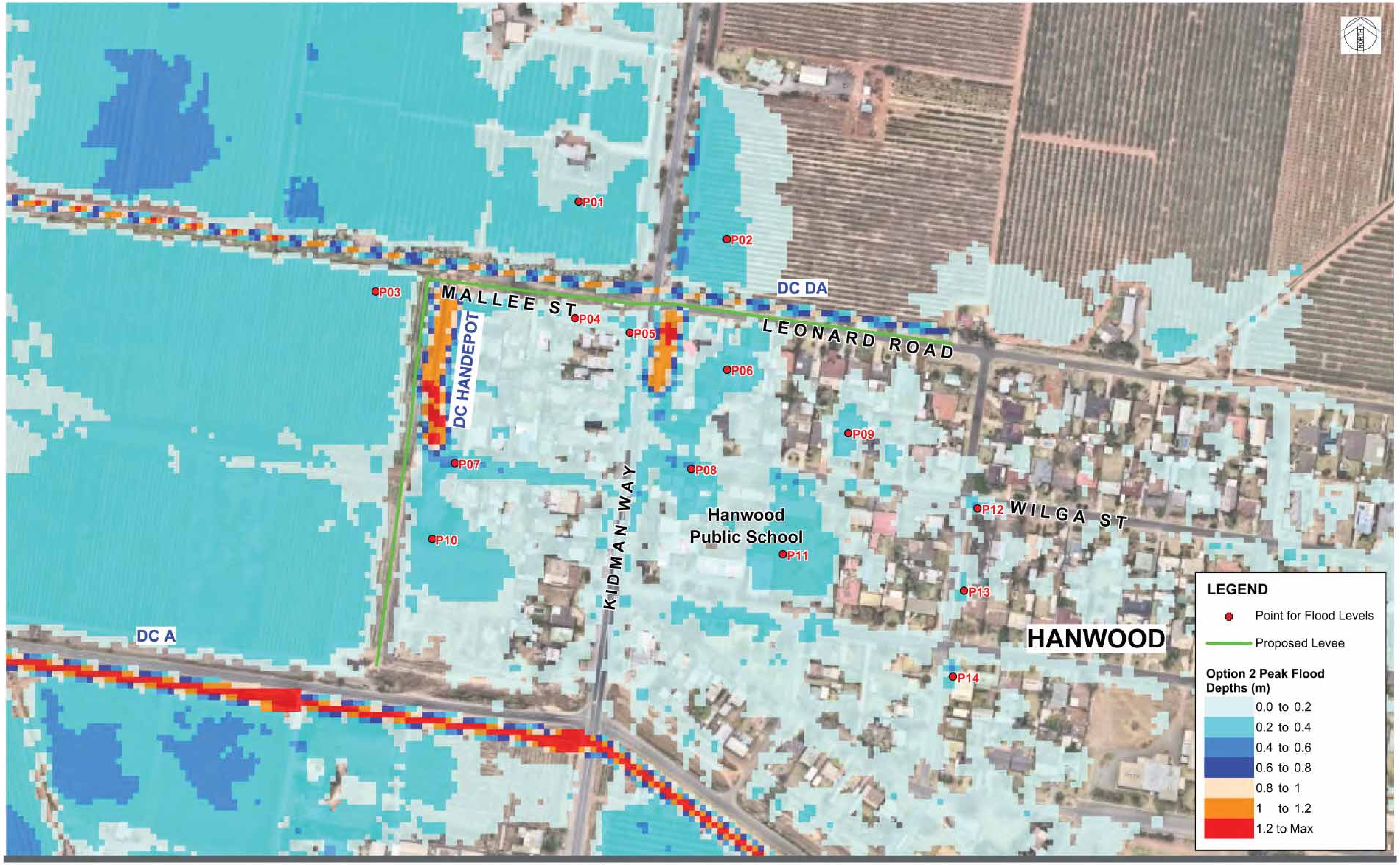

Option 1 (Full Levee, no pump), 1% AEP Event, 12 hr Storm Duration

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A2

FigA2_Opt1_100yr12hr_depth.wor_r1, REV NO: 1

Date
20/07/2018

Size
A3

Scale
0 120
metres

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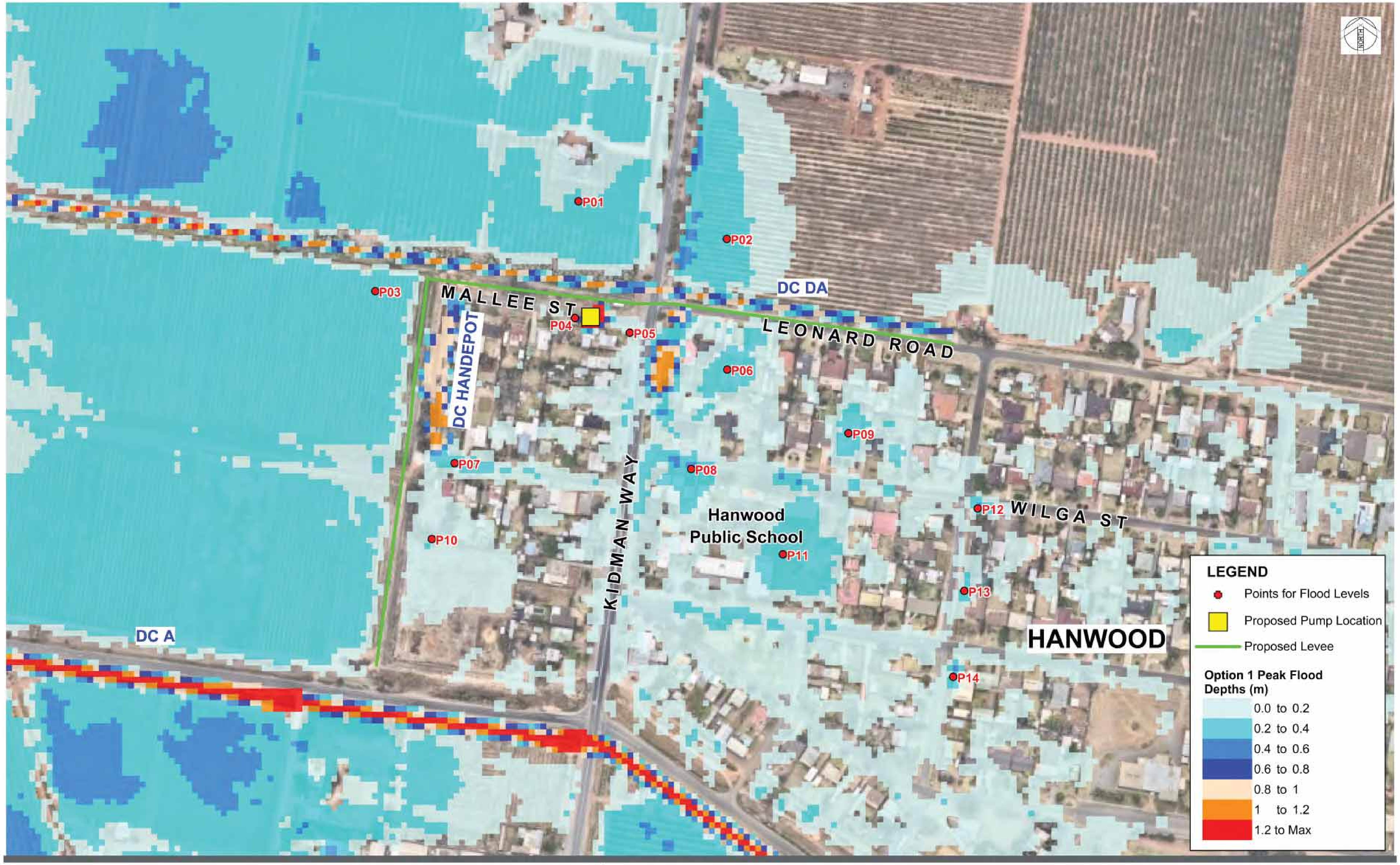

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Date
20/07/2018

Size
A3

Scale
0 120
metres

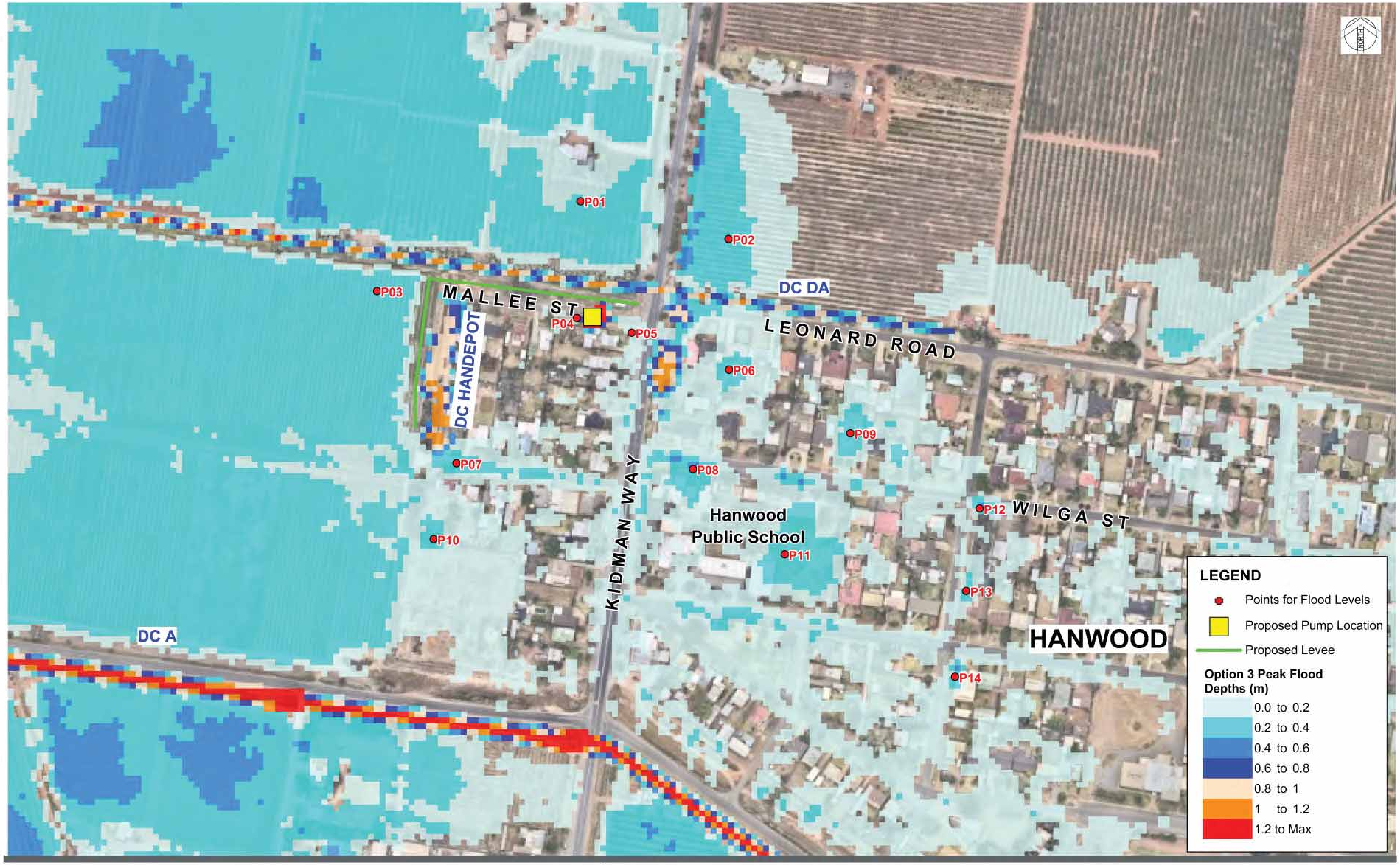
Option 2 (Full Levee, 9m3/s pump), 1% AEP Event, 12 hr Storm

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A3

FigA3_Opt2_100yr12hr_depth_r1.wor, REV NO: 1



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Date
13/07/2018

Size
A3

Scale
0 120
metres

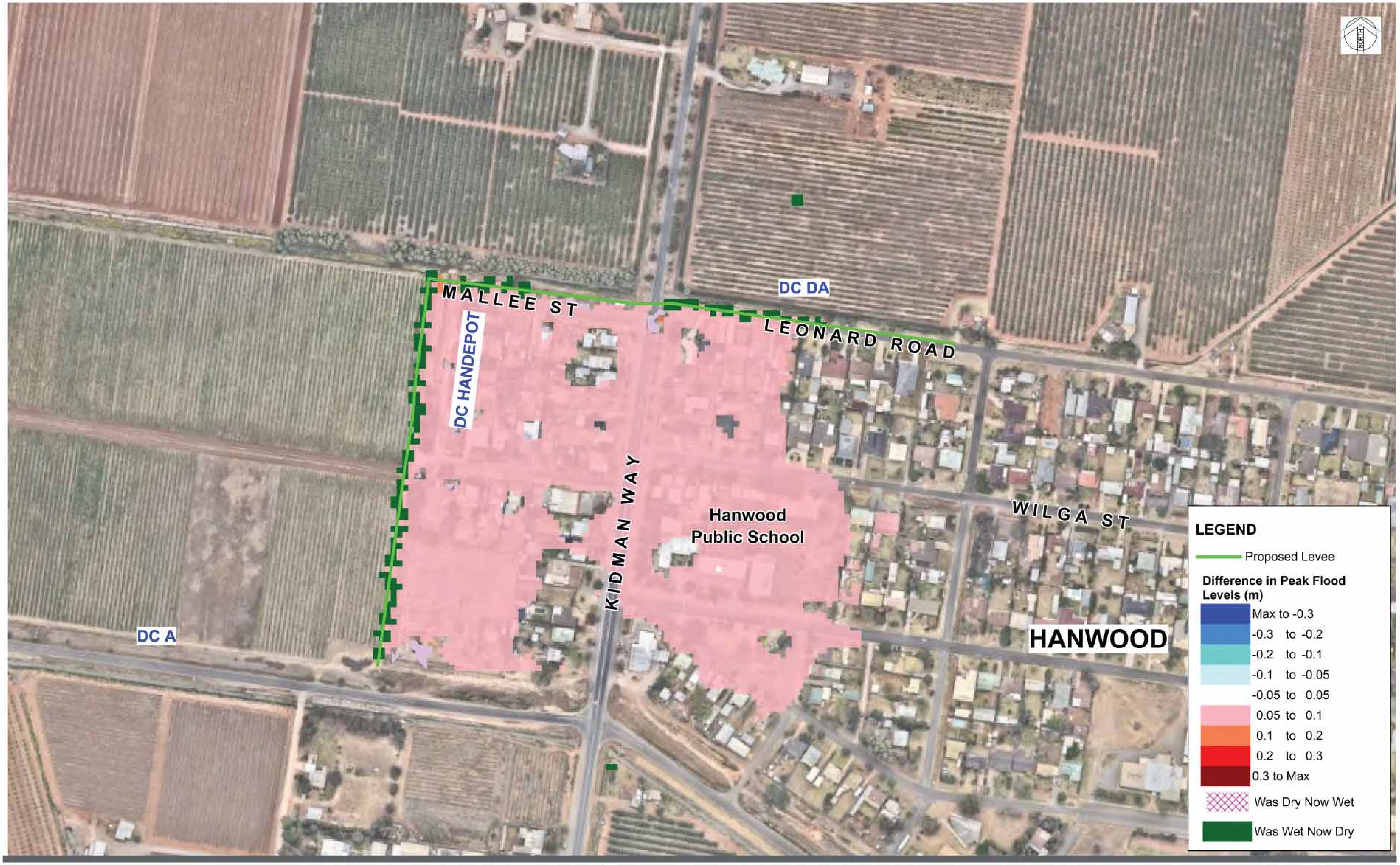
Option 3 (Short Levee, 9m3/s pump), 1% AEP Event, 12 hr Storm

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A4

FigA4_Opt3_100yr12hr_depth.wor, REV NO: 0



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Date
20/07/2018

Size
A3

Scale
0 120
metres

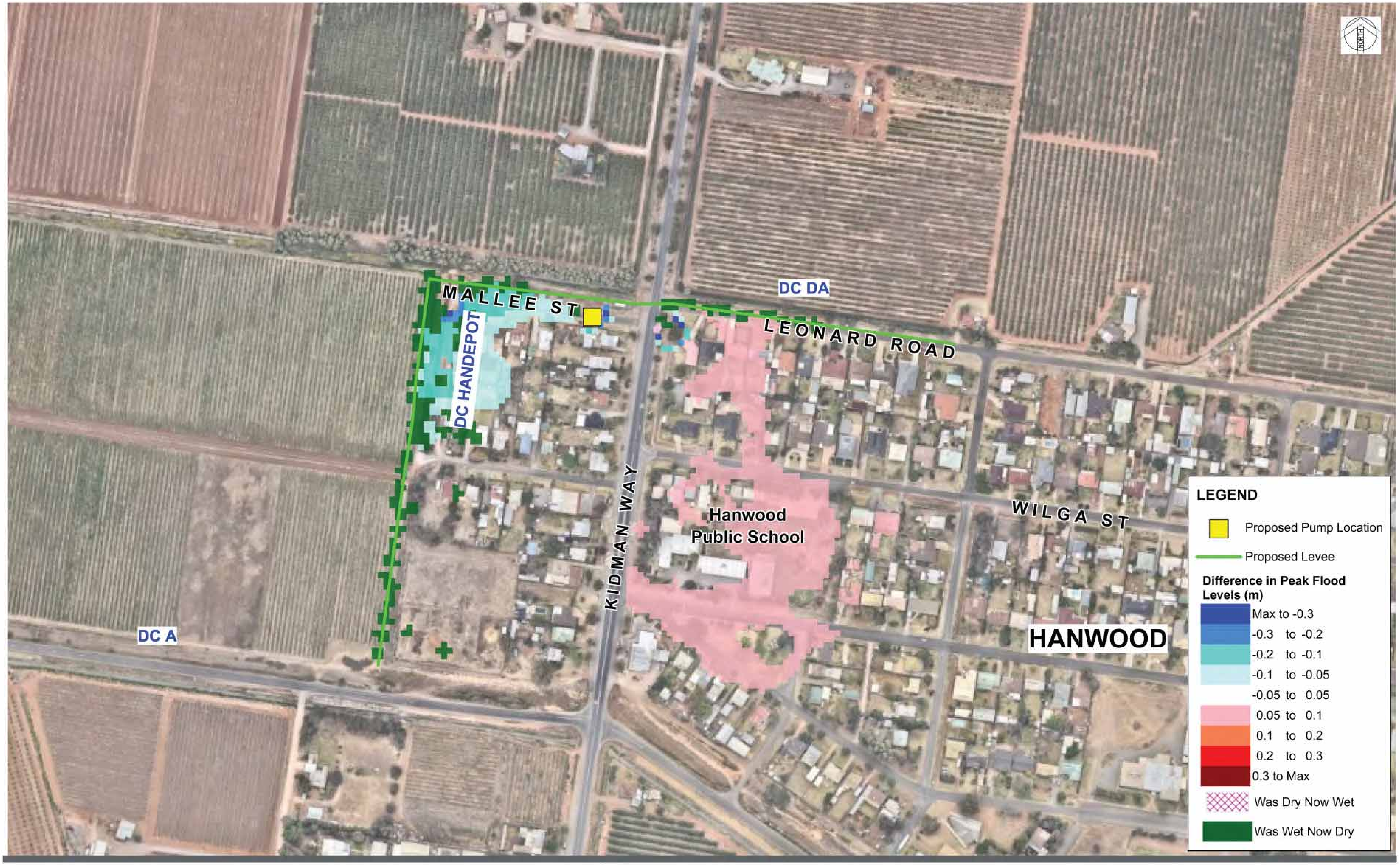
Option 1 (Full Levee, no pump) vs Existing, 1% AEP Event, 12 hr

Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A5

FigA5_Opt1_vs_Exis_100yr12hr_level_r1.wor, REV NO: 1



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Date
20/07/2018

Size
A3

Scale
0 120
metres

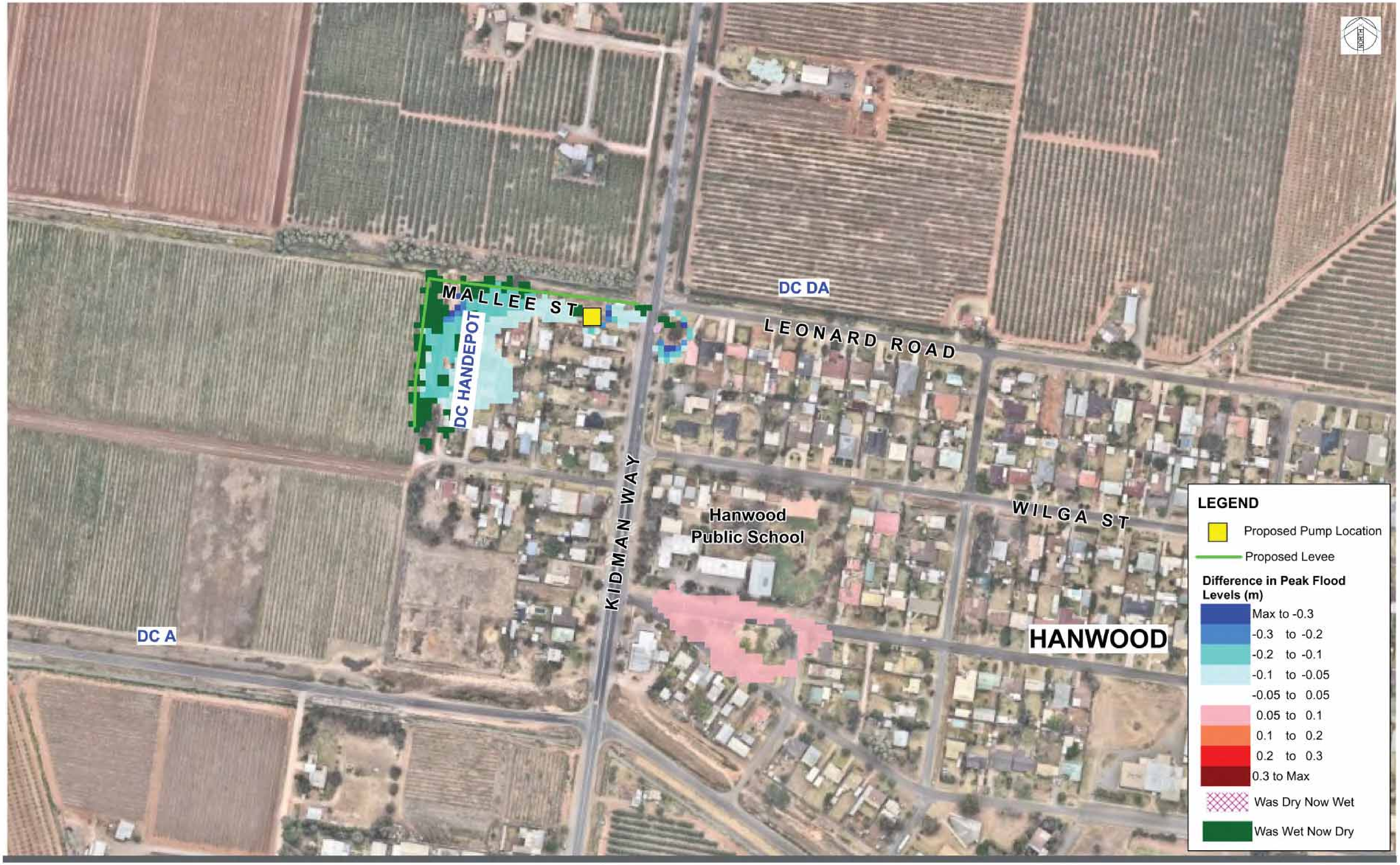
Option 2 (Full Levee, 9m3/s pump) vs Existing, 1% AEP Event, 12 hr

Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A6

FigA6_Opt2_vs_Exis_100yr12hr_level_r1.wor, REV NO: 1



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Date
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Size
A3

Scale
0 120
metres

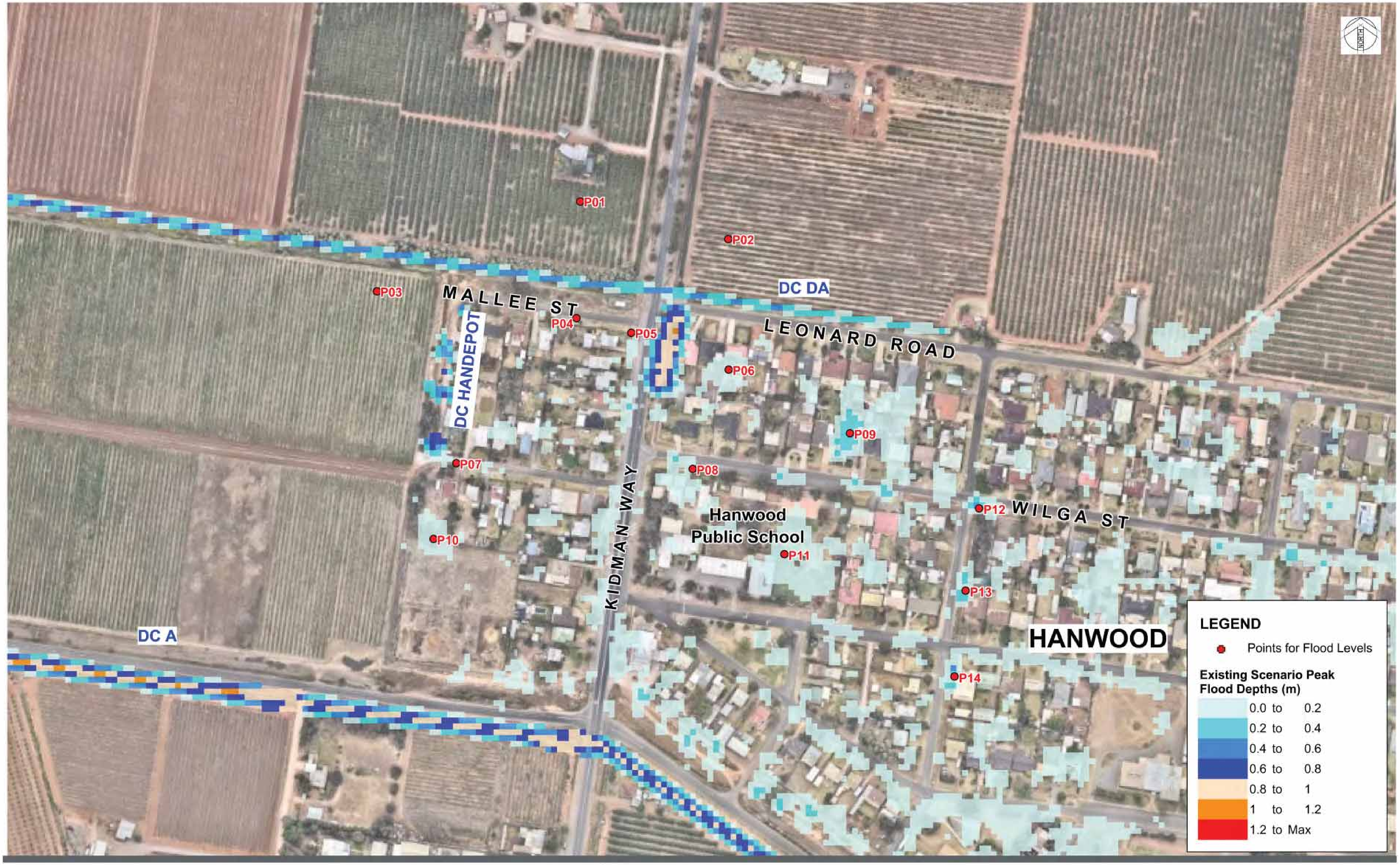
Option 3 (Short Levee, 9m3/s pump) vs Existing, 1% AEP Event, 12 hr

Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A7

FigA7_Opt3_vs_Exis_100yr12hr_level.wor, REV NO: 0



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Date
16/07/2018

Size
A3

Scale
0 120
metres

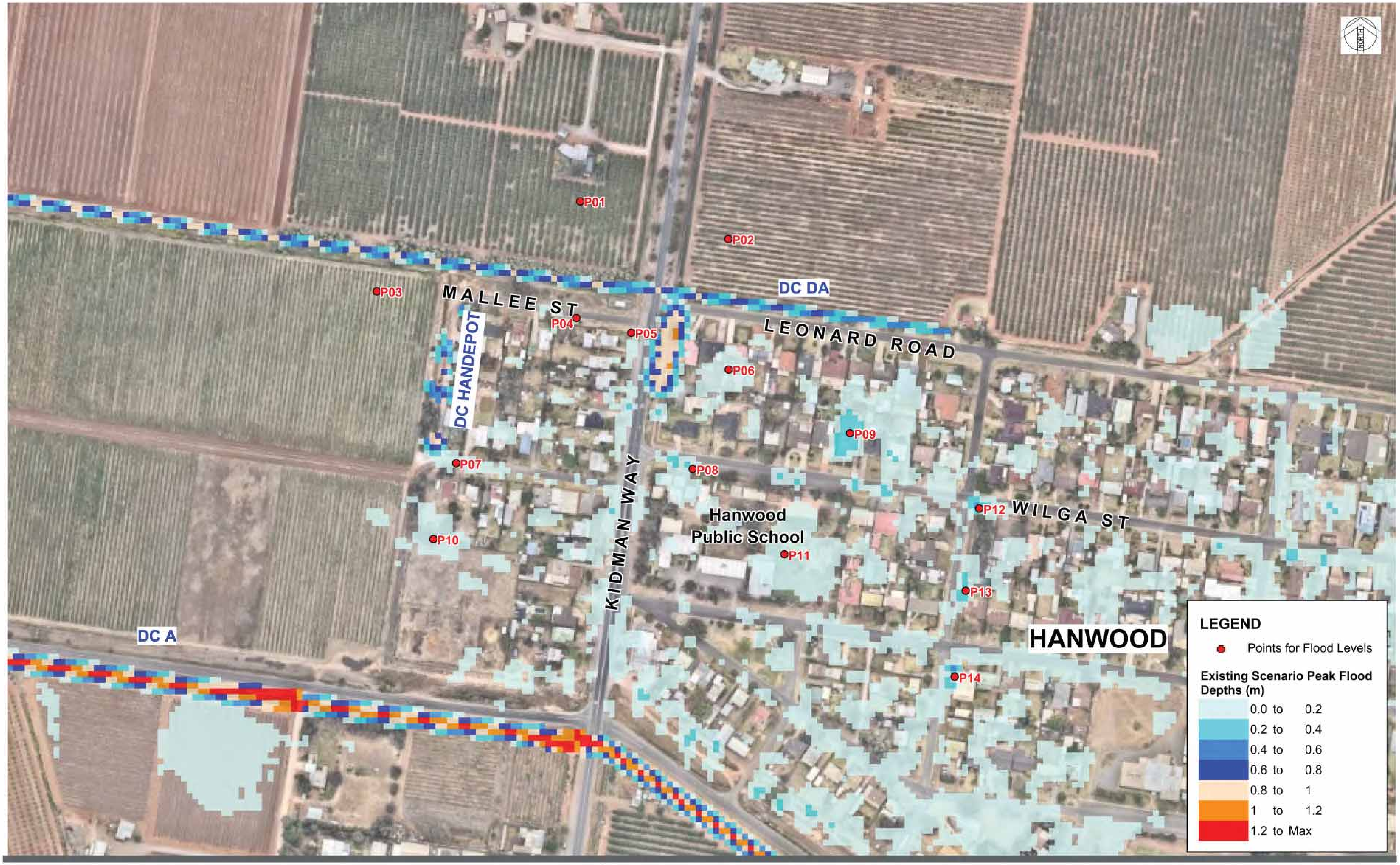
Existing Scenario, 1% AEP Event, 20 min Storm Duration

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A8

FigA8_Existing_100yr20min_depth.wor, REV NO: 0



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Date
20/07/2018

Size
A3

Scale
0 120
metres

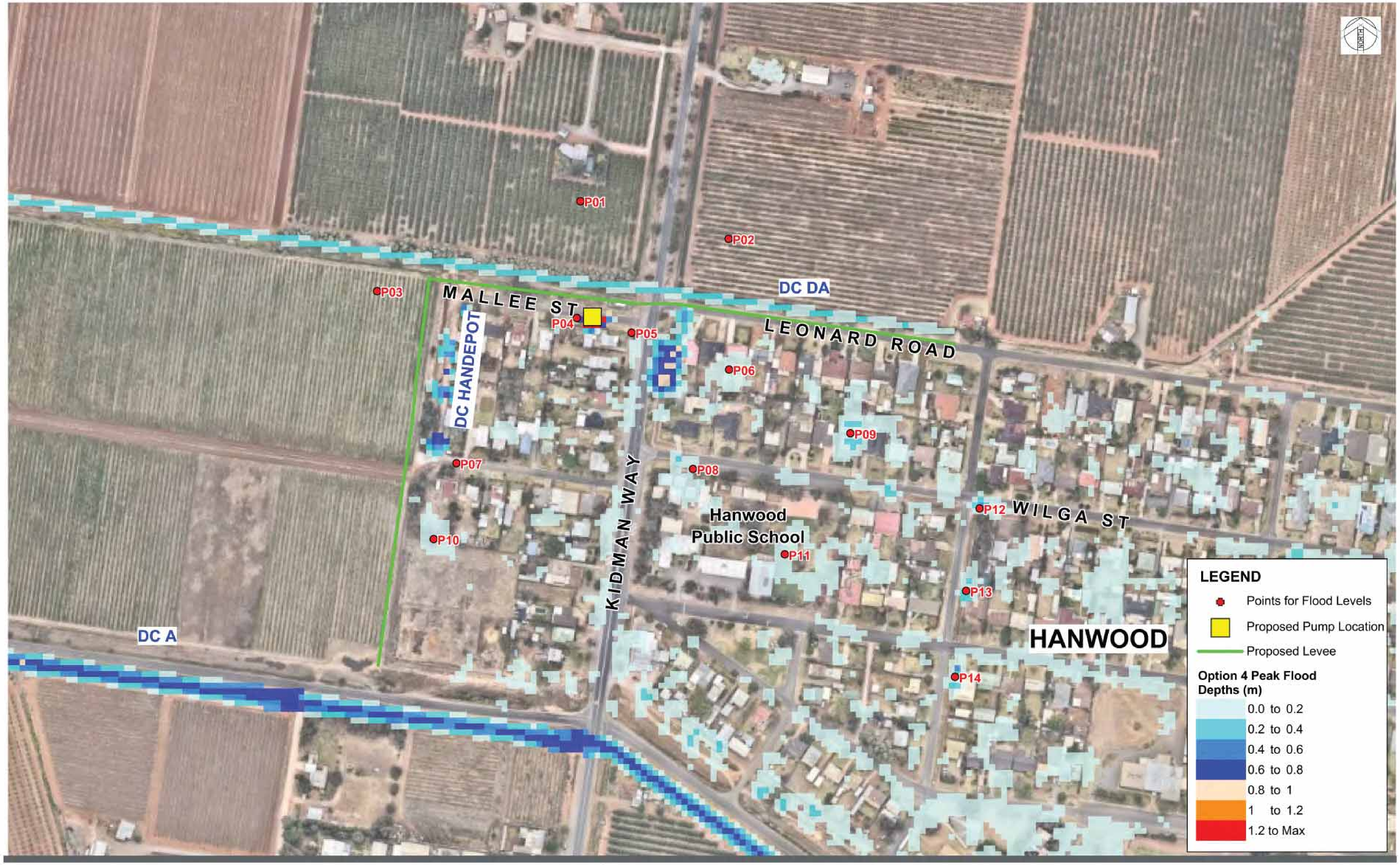
Existing Scenario, 1% AEP Event, 60 min Storm Duration

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A9

FigA9_Existing_100yr60min_depth.wor, REV NO: 0



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Date
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Size
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Scale
0 120
metres

Option 4 (Full Levee, 9m3/s pump), 1% AEP Event, 20 min Storm

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A10
FigA10_Opt4_100yr20min_depth.wor, REV NO: 0

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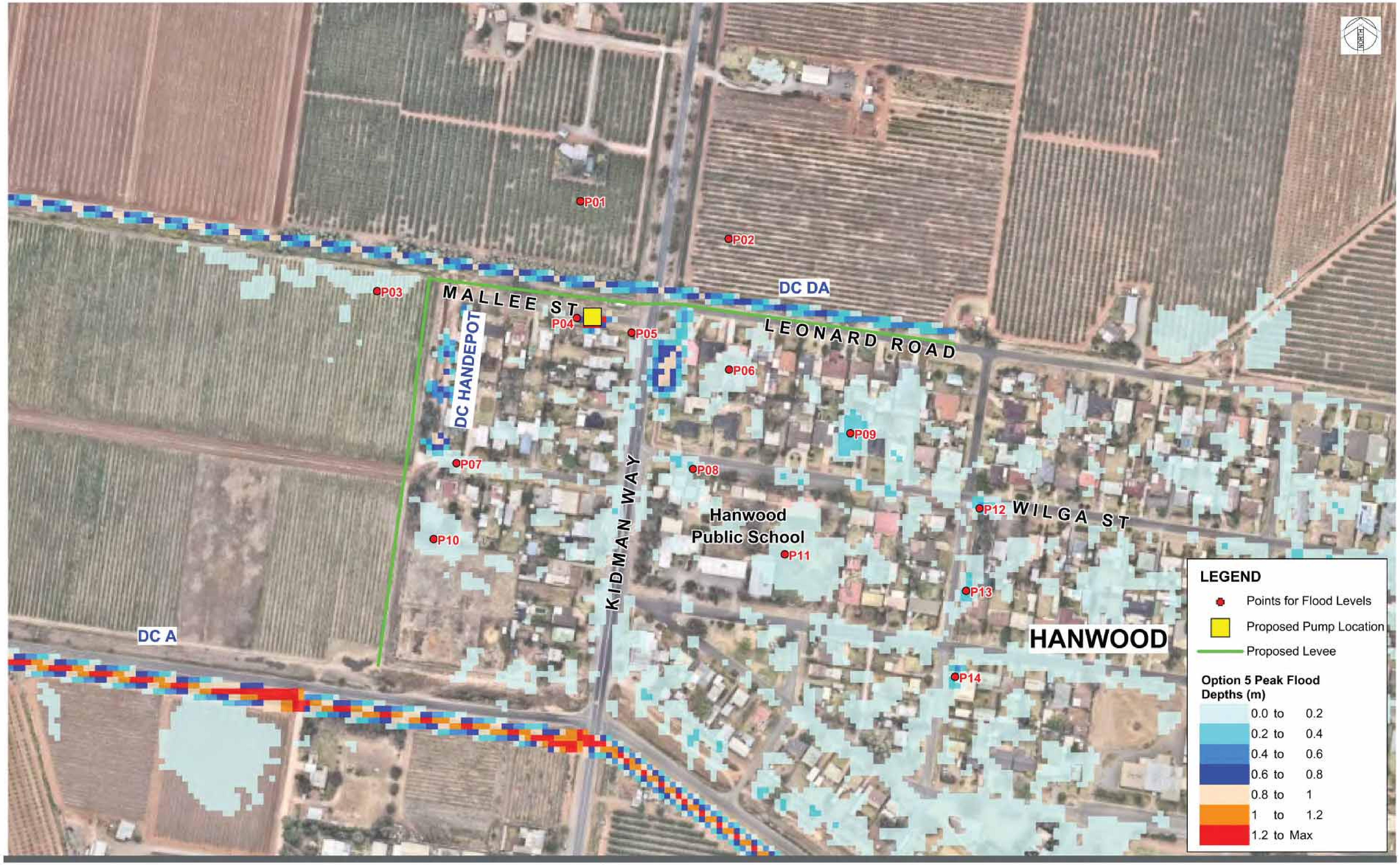

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Size
A3

Scale
0 120
metres

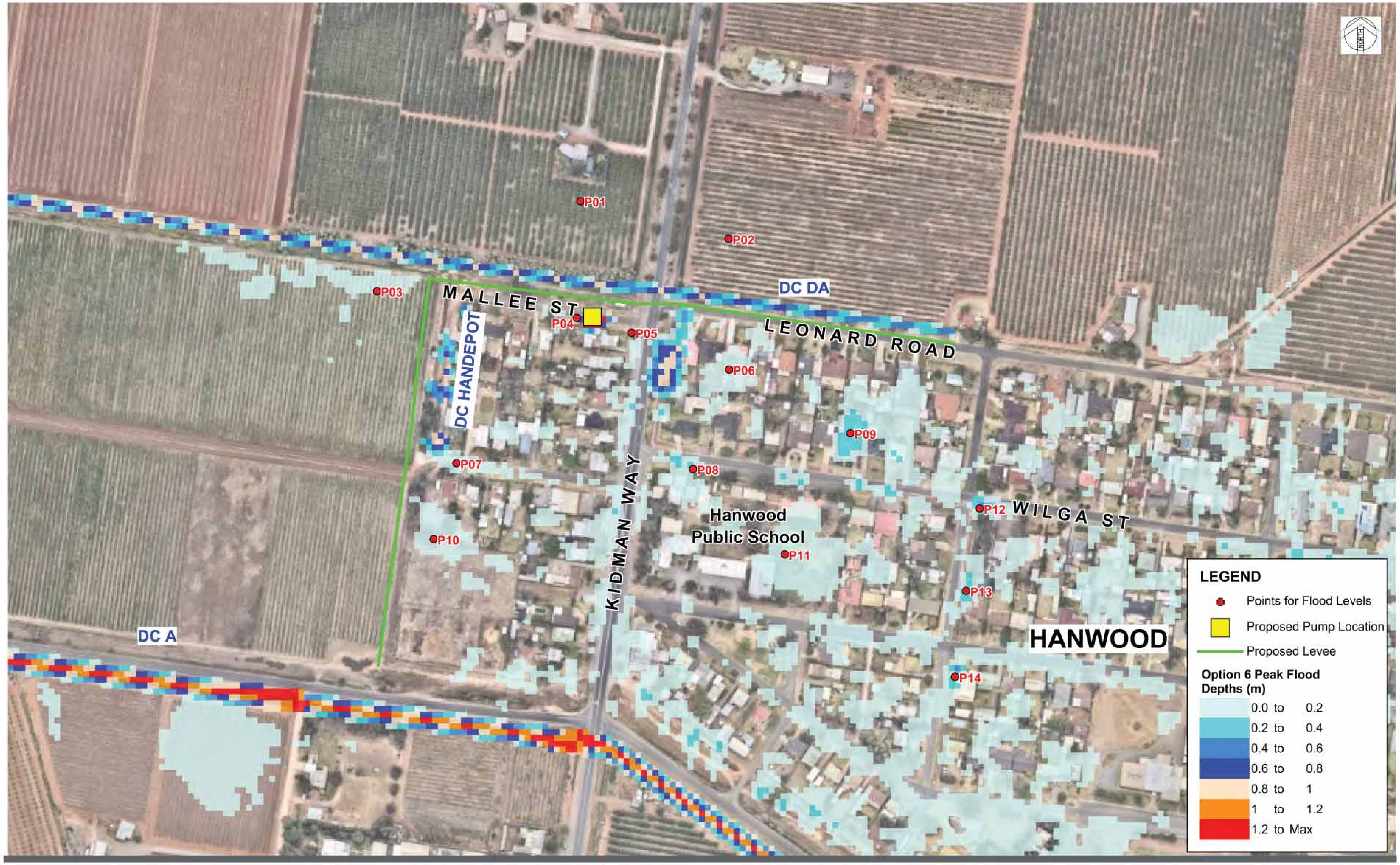
Option 5 (Full Levee, 5.85m3/s pump), 1% AEP Event, 60 min Storm

Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A11

FigA11_Opt5_100yr60min_depth.wor, REV NO: 0



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Size
A3

Scale
0 120
metres

Option 6 (Full Levee, 3.5m3/s pump), 1% AEP Event, 60 min Storm
Peak Flood Depths
Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A12
FigA12_Opt6_100yr60min_depth.wor, REV NO: 0

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Size
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Scale
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metres

Option 4 (Full Levee, 9m3/s pump) vs Existing, 1% AEP Event, 20 min
Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A13
FigA13_Opt4_vs_Exis_100yr20min_level.wor, REV NO: 0

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Size
A3

Scale
0 120
metres

Option 5 (Full Levee, 5.85m3/s pump) vs Existing, 1% AEP Event, 60 min Storm

Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump

FIGURE A14

Fig14_Opt5_vs_Exis_100yr60min_level.wor, REV NO: 0

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Date
20/07/2018

Size
A3

Scale
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metres

Option 6 (Full Levee, 3.5m3/s pump) vs Existing, 1% AEP Event, 60 min Storm

Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A15

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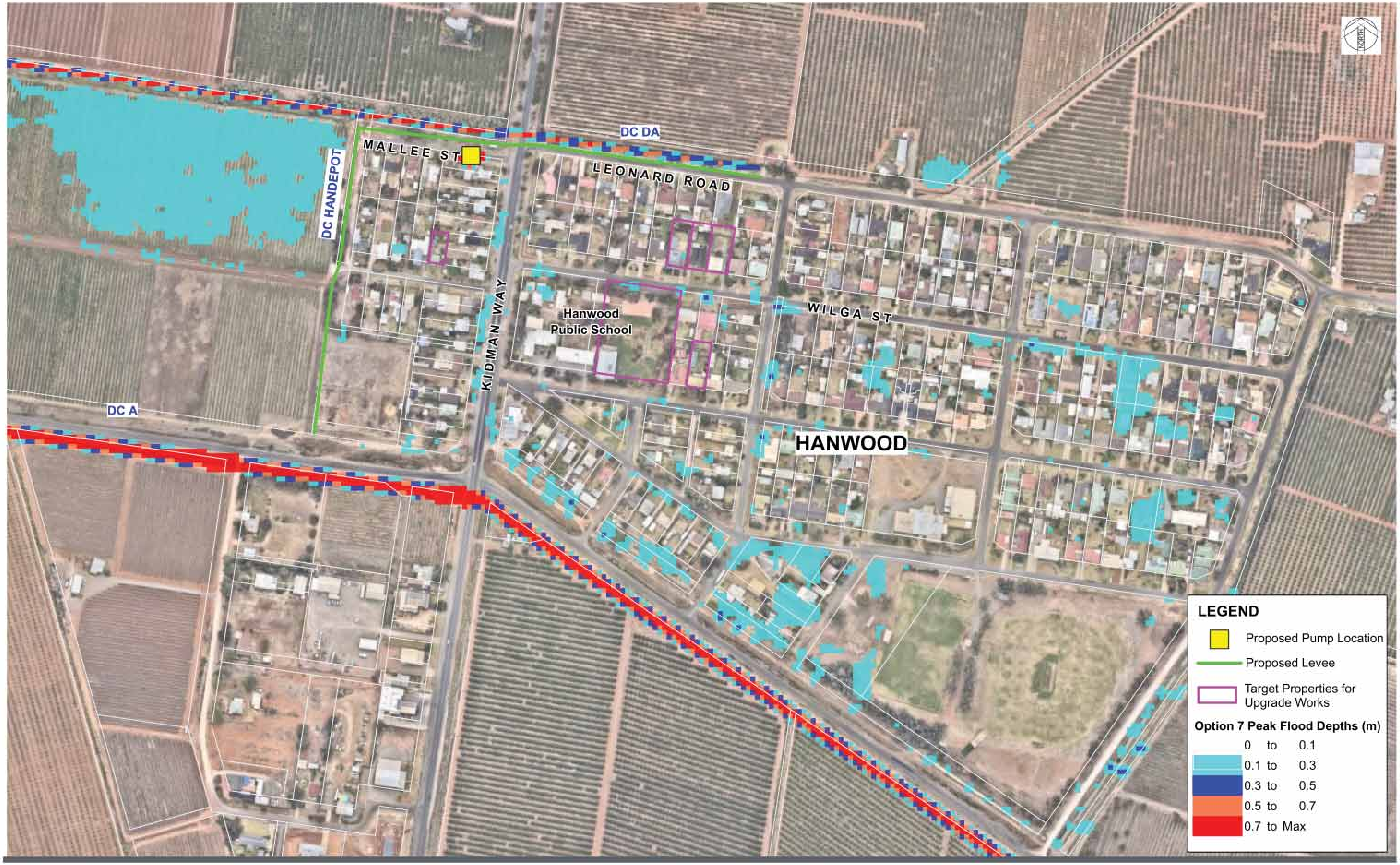

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Date
04/12/2018

Size
A3

Scale
0 150

meters

Proposed Township Overland Flow System Upgrade
Option 7 (Full Levee, 1.0m3/s pump), 1% AEP Event, 60 min Storm Duration
Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A16
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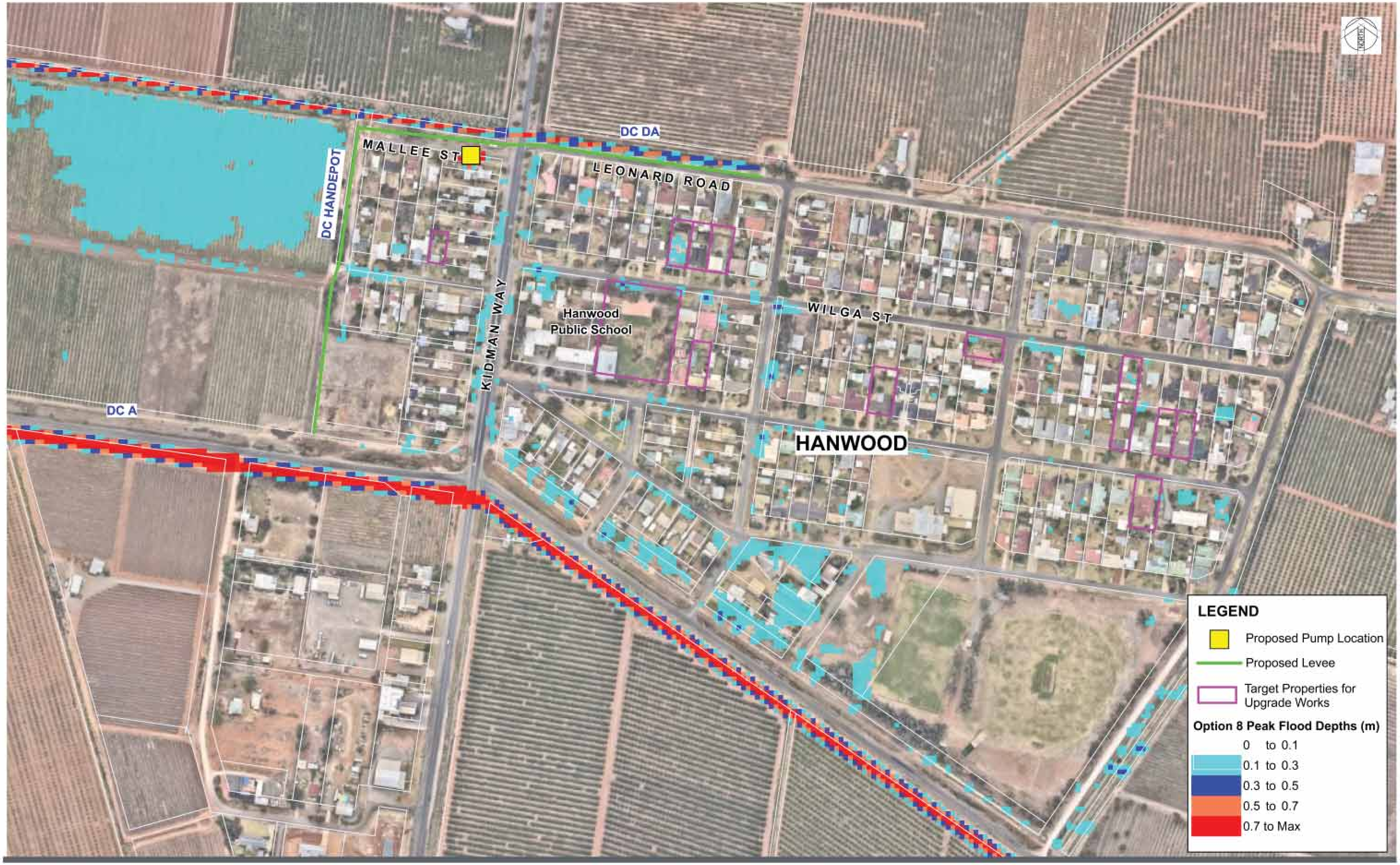

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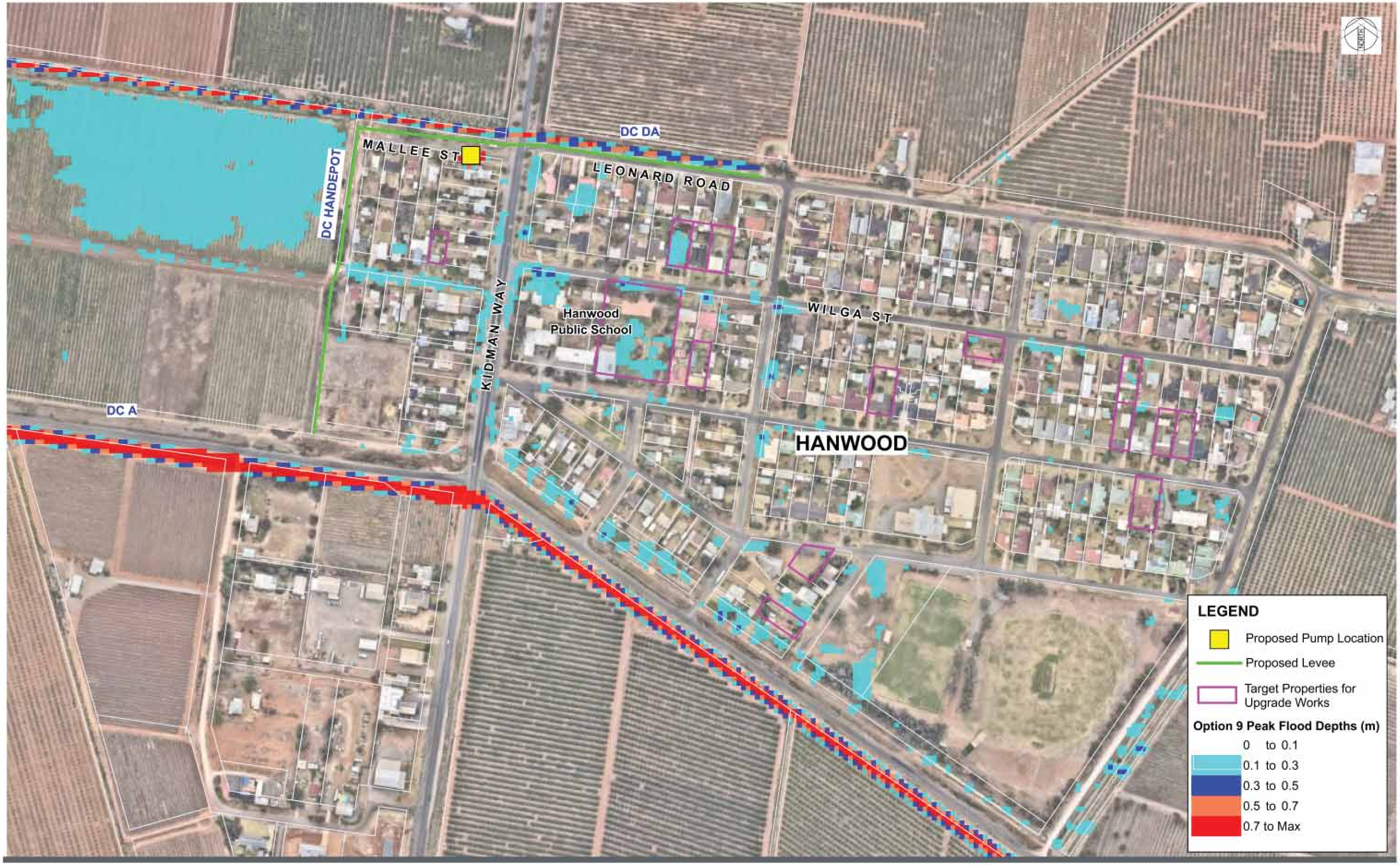
Date
04/12/2018

Size
A3

Scale
0 150
meters

Proposed Township Overland Flow System Upgrade
Option 8 (Full Levee, 1.0m3/s pump), 1% AEP Event, 60 min Storm Duration
Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A17
FigA17_Opt8_100yr60min_depth.wor, REV NO: 0



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Date
04/12/2018

Size
A3

Scale
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meters

Proposed Township Overland Flow System Upgrade
Option 9 (Full Levee, 1.0m3/s pump), 1% AEP Event, 60 min Storm Duration
Peak Flood Depths

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A18
FigA18_Opt9_100yr60min_depth.wor, REV NO: 0

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Date
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Size
A3

Scale
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meters

Proposed Township Overland Flow System Upgrade
Option 7 (Full Levee, 1.0m3/s pump) vs Existing, 1% AEP 60min Storm Duration
Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A19
FigA19_Opt7_vs_Exis_100yr60min_level.wor, REV NO: 0

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Date
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Size
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Scale
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meters

Proposed Township Overland Flow System Upgrade
Option 8 (Full Levee, 1.0m3/s pump) vs Existing, 1% AEP 60min Storm Duration
Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A20
FigA20_Opt8_vs_Exis_100yr60min_level.wor, REV NO: 0

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Date
04/12/2018

Size
A3

Scale
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meters

Proposed Township Overland Flow System Upgrade
Option 9 (Full Levee, 1.0m3/s pump) vs Existing, 1% AEP 60min Storm Duration
Difference in Peak Flood Levels

Detail Design and Investigation of Hanwood Levee and Stormwater Pump
FIGURE A21
FigA21_Opt9_vs_Exis_100yr60min_level.wor, REV NO: 0

APPENDIX

B

PRELIMINARY ENGINEERING DRAWINGS

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GRIFFITH CITY COUNCIL

HANWOOD STORMWATER PUMP AND LEVEE CONCEPT DESIGN OPTIONS

COVER SHEET
DRAWING LIST
LOCALITY MAP

Map section reproduced with permission of NEARMAP

LOCALITY PLAN
SCALE 1:2000

CIVIL DRAWINGS

NUMBER	TITLE
80518062-CI-100	COVER SHEET, DRAWING LIST & LOCALITY MAP
80518062-CI-105	EXISTING SITE PLAN
80518062-CI-110	CONCEPT OPTION 1
80518062-CI-115	CONCEPT OPTION 2
80518062-CI-120	CONCEPT OPTION 3
80518062-CI-130	CONCEPT INTERSECTION RAISING - PLAN
80518062-CI-135	CONCEPT INTERSECTION RAISING - LONGSECTION
80518062-CI-140	CONCEPT INTERSECTION RAISING - TYPICAL CROSS SECTIONS

0 50m
SCALE 1:2000 @A1

REVISION IN PROGRESS

Rev.	Date	Description	Des.	Verif.	Appd.
1	-	CONCEPT DESIGN ISSUE	PW	SGB	-

Drawn LDB	Date AUG/18
Checked SGB	Date AUG/18
Designed PW	Date AUG/18
Verified SGB	Date AUG/18
Approved -	- -

Status
PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION PURPOSES

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Datum	Register	Scale	Size
	-	1:2000	A1

Drawing Number	Revision
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PLAN
SCALE 1:1000

Rev.	Date	Description	Des.	Verif.	Appd.
1	-	CONCEPT DESIGN ISSUE	PW	SGB	-

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SCALE 1:1000 @A1

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Drawn	LDB	AUG/18
Checked	SGB	AUG/18
Designed	PW	AUG/18
Verified	SGB	AUG/18
Approved	-	-

Client GRIFFITH CITY COUNCIL

Project HANWOOD STORMWATER
PUMP AND LEVEE
CONCEPT DESIGN OPTIONS

Title EXISTING SITE PLAN

Status PRELIMINARY
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PLAN
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Client GRIFFITH CITY COUNCIL

Project HANWOOD STORMWATER
PUMP AND LEVEE
CONCEPT DESIGN OPTIONS

Title CONCEPT OPTION 1

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Drawing Number	Revision
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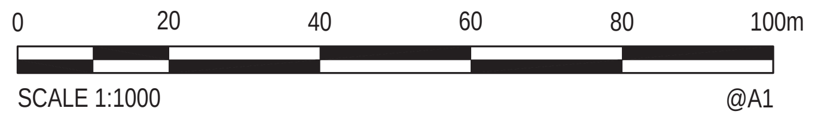

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PUMP AND LEVEE
CONCEPT DESIGN OPTIONS

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Project HANWOOD STORMWATER
PUMP AND LEVEE
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Project HANWOOD STORMWATER
PUMP AND LEVEE
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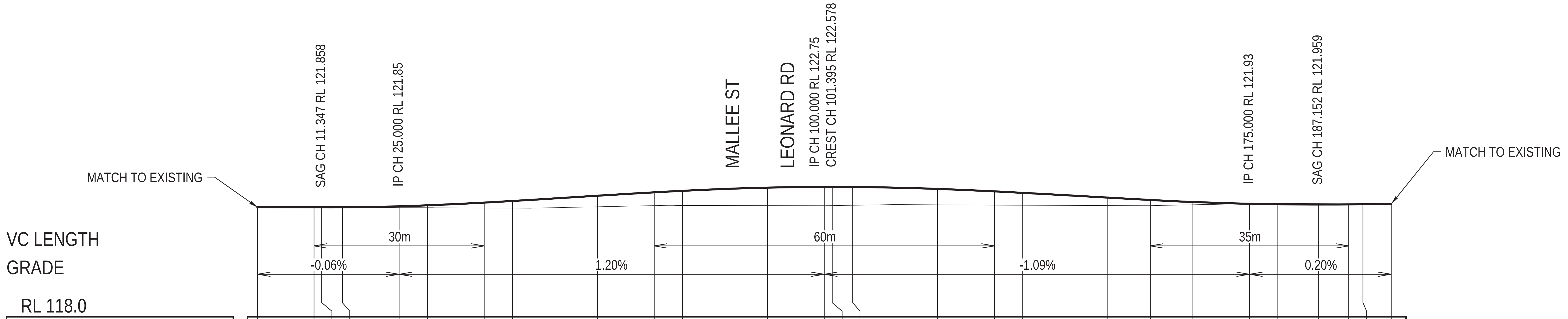

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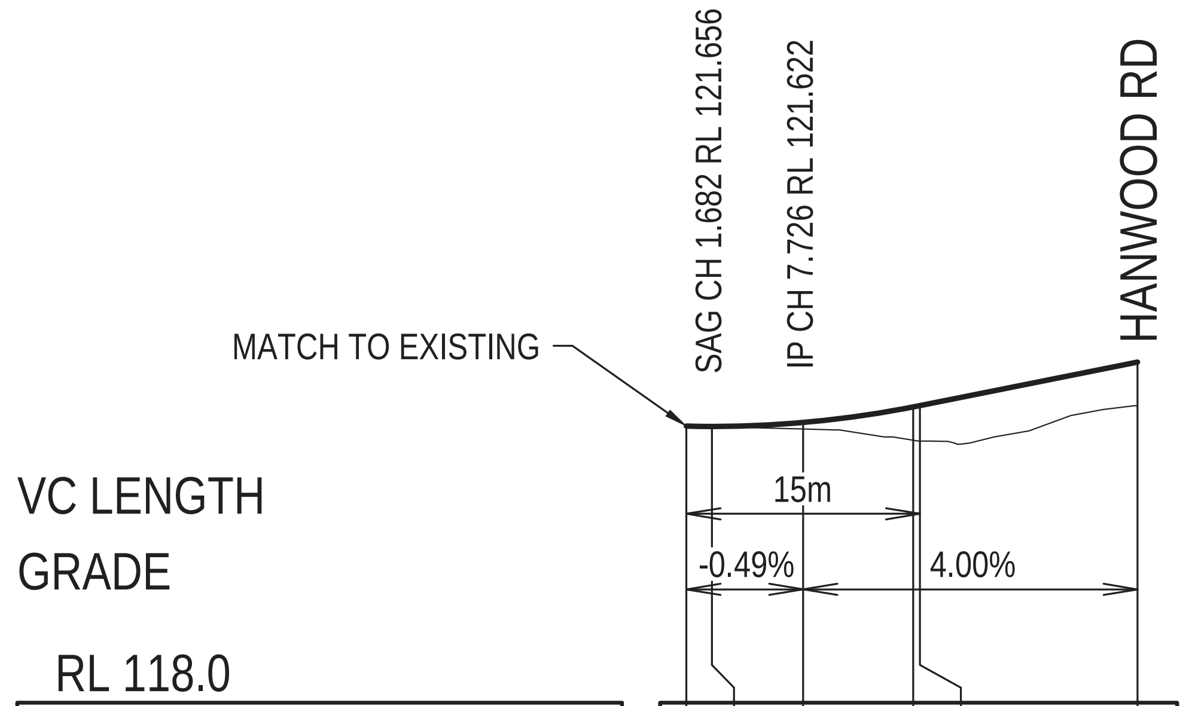
DESIGN SURFACE	121.864	121.858	121.858	121.861	121.897	121.931	122.030	122.090	122.270	122.390	122.445	122.554	122.578	122.578	122.576	122.512	122.422	122.367	122.203	122.121	122.050	121.986	121.969	121.959	121.965	121.969	121.979
NATURAL SURFACE	121.864	121.858	121.857	121.855	121.849	121.846	121.836	121.833	121.879	121.920	121.928	121.922	121.918	121.923	121.935	121.952	121.940	121.935	121.925	121.922	121.951	121.991	122.001	121.992	121.986	121.983	121.979
CHAINAGE	0.000	10.000	11.347	15.000	25.000	30.000	40.000	45.000	60.000	70.000	75.000	90.000	100.000	101.395	105.000	120.000	130.000	135.000	150.000	157.500	165.000	175.000	180.000	187.152	192.500	195.000	200.000

LONGITUDINAL SECTION - HANWOOD ROAD

SCALE H 1:500, V 1:100



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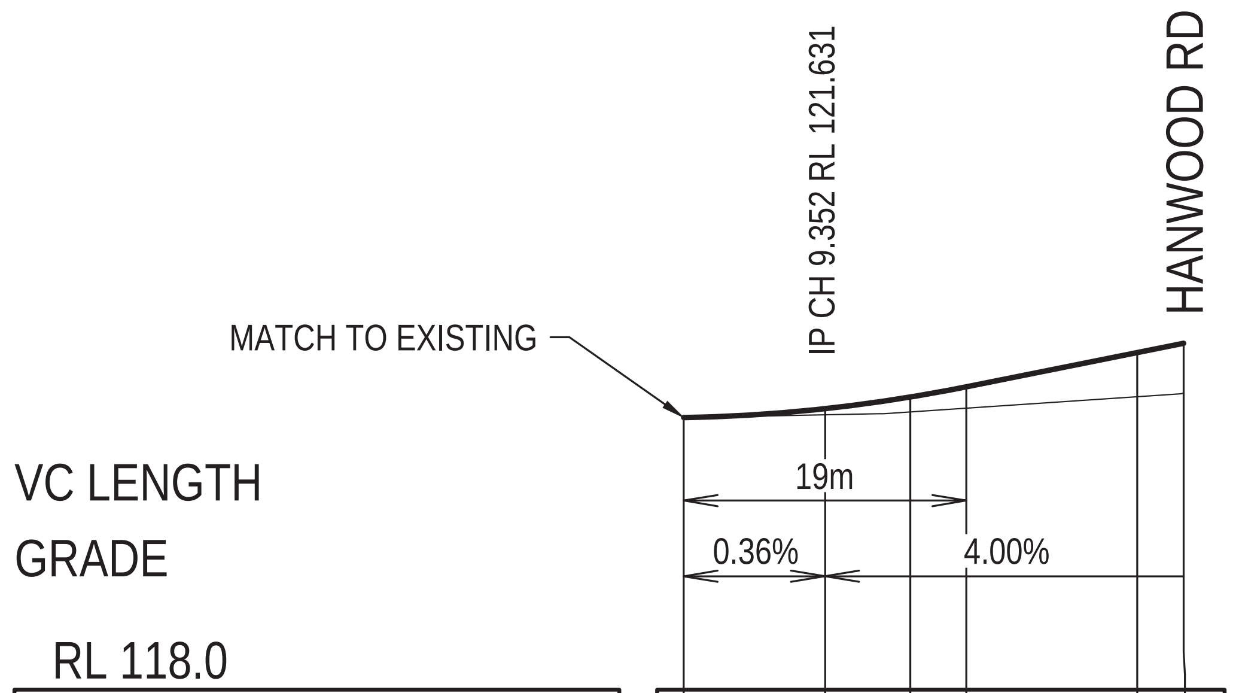
DESIGN SURFACE	121.660	121.656	121.709	121.914	121.931	122.507
NATURAL SURFACE	121.660	121.652	121.624	121.473	121.462	121.925
CHAINAGE	0.000	1.682	7.726	15.000	15.452	29.832

LONGITUDINAL SECTION - MALLEE STREET

SCALE H 1:500, V 1:100



CUT/FILL DEPTH	-0.001	0.082	0.196	0.282	0.583	0.660
DESIGN SURFACE	121.597	121.716	121.870	122.005	122.457	122.580
NATURAL SURFACE	121.599	121.634	121.674	121.723	121.874	121.920
CHAINAGE	0.000	9.352	15.000	18.704	30.000	33.066

LONGITUDINAL SECTION - LEONARD ROAD

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Client GRIFFITH CITY COUNCIL

Project HANWOOD STORMWATER
PUMP AND LEVEE
CONCEPT DESIGN OPTIONS

Title
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LONGSECTION

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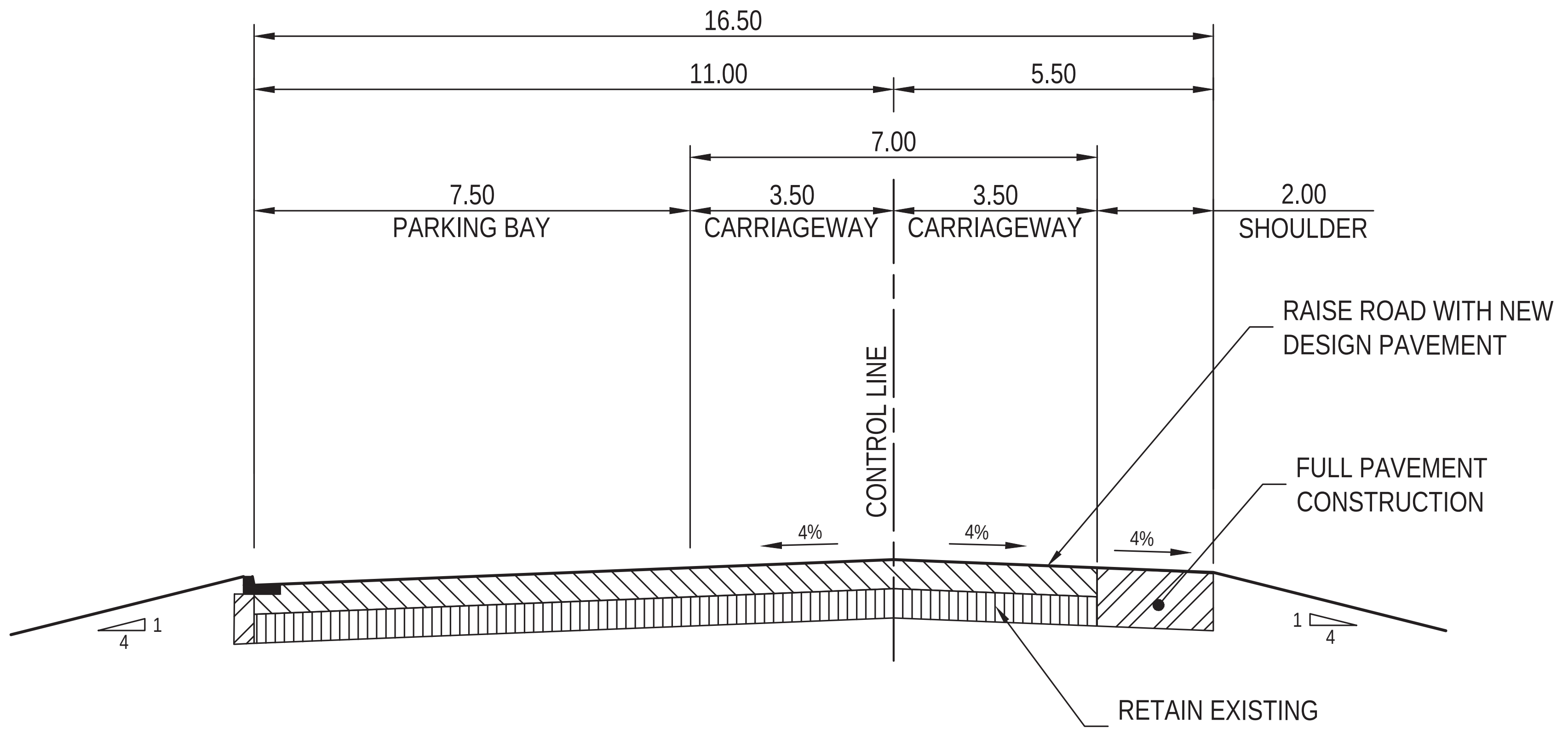

TYPICAL CROSS SECTION
HANWOOD AVENUE

SCALE 1:100

SECTION 1-1
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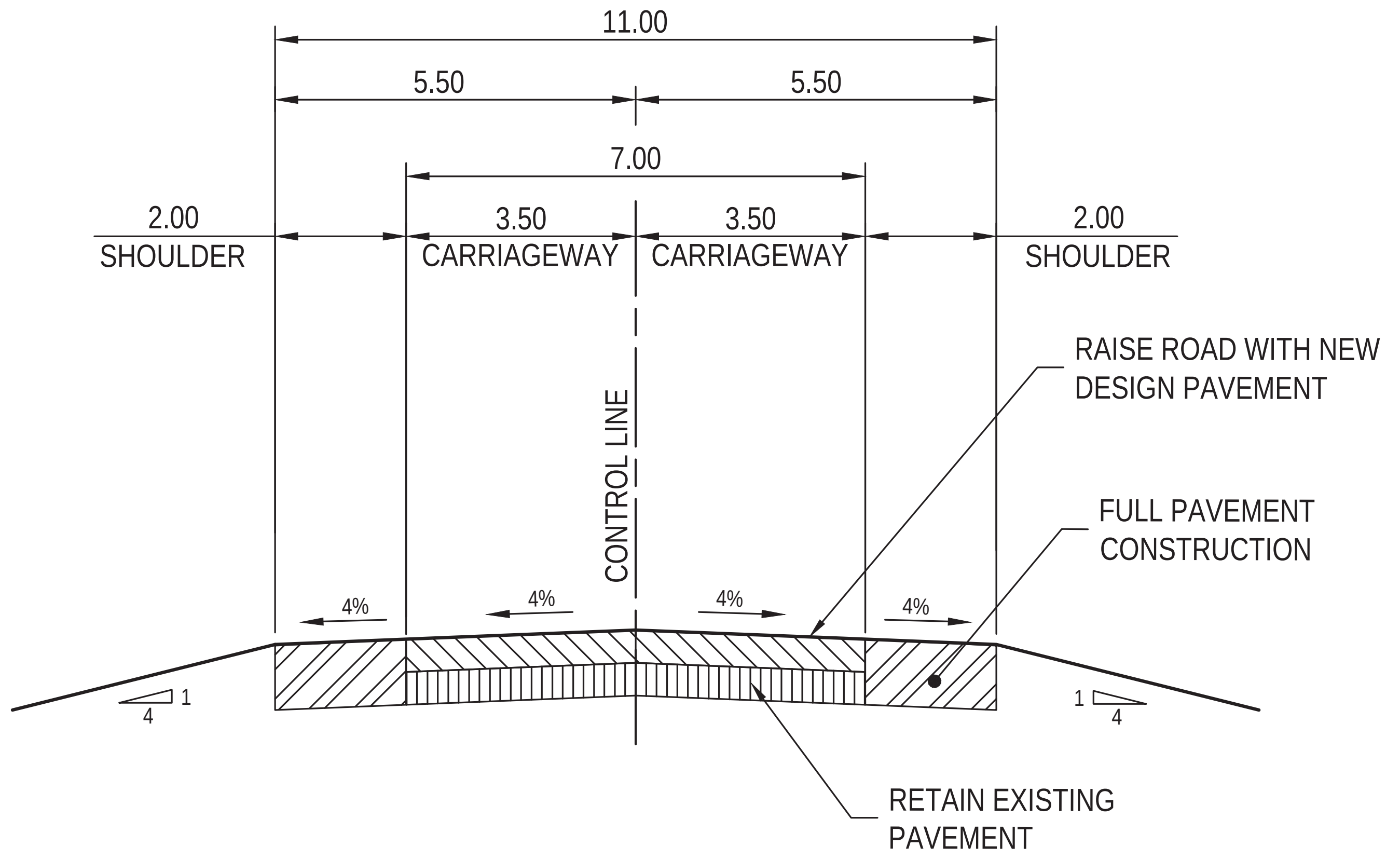

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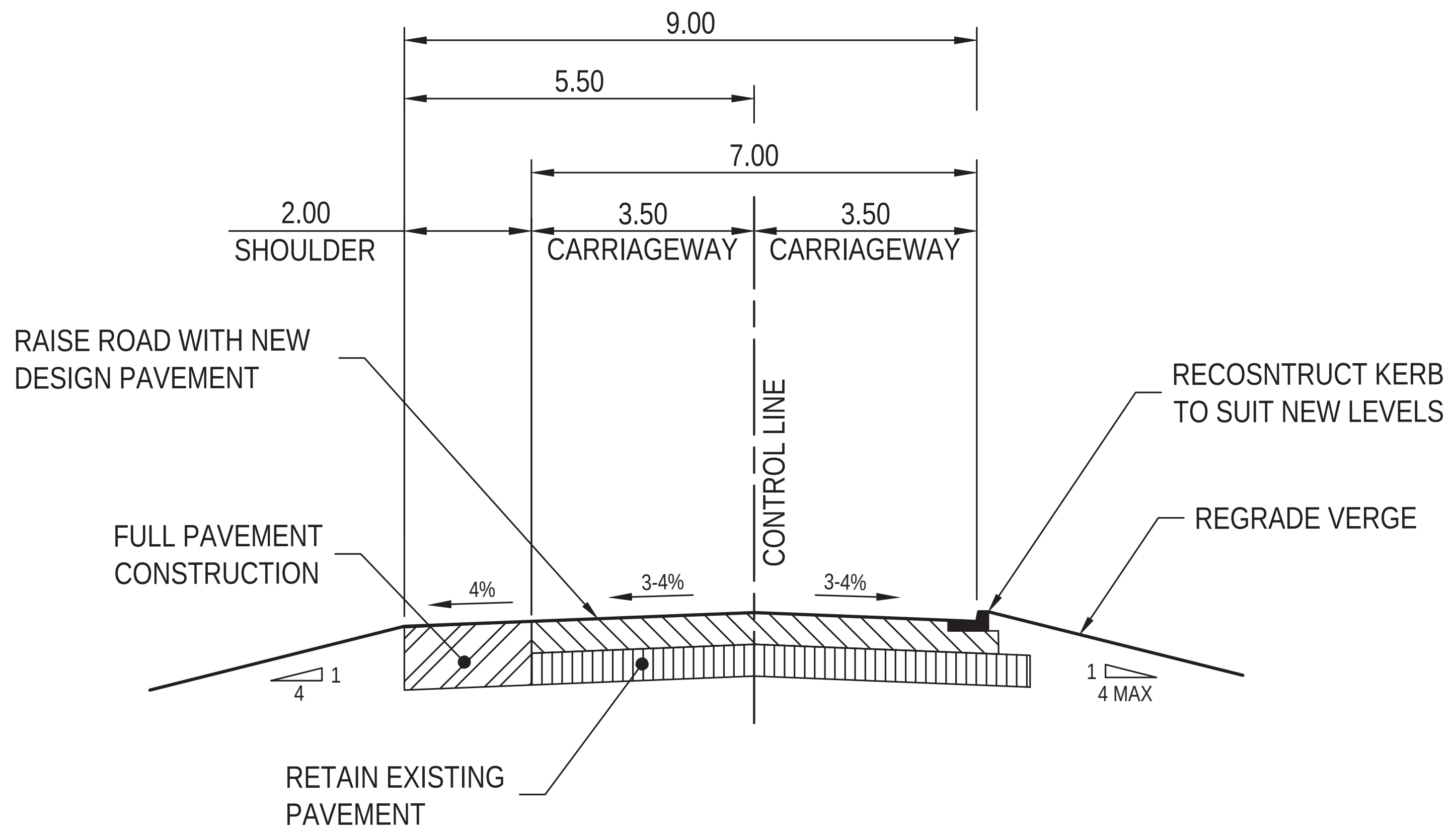

TYPICAL CROSS SECTION
MALLEE STREET

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SECTION 3-3
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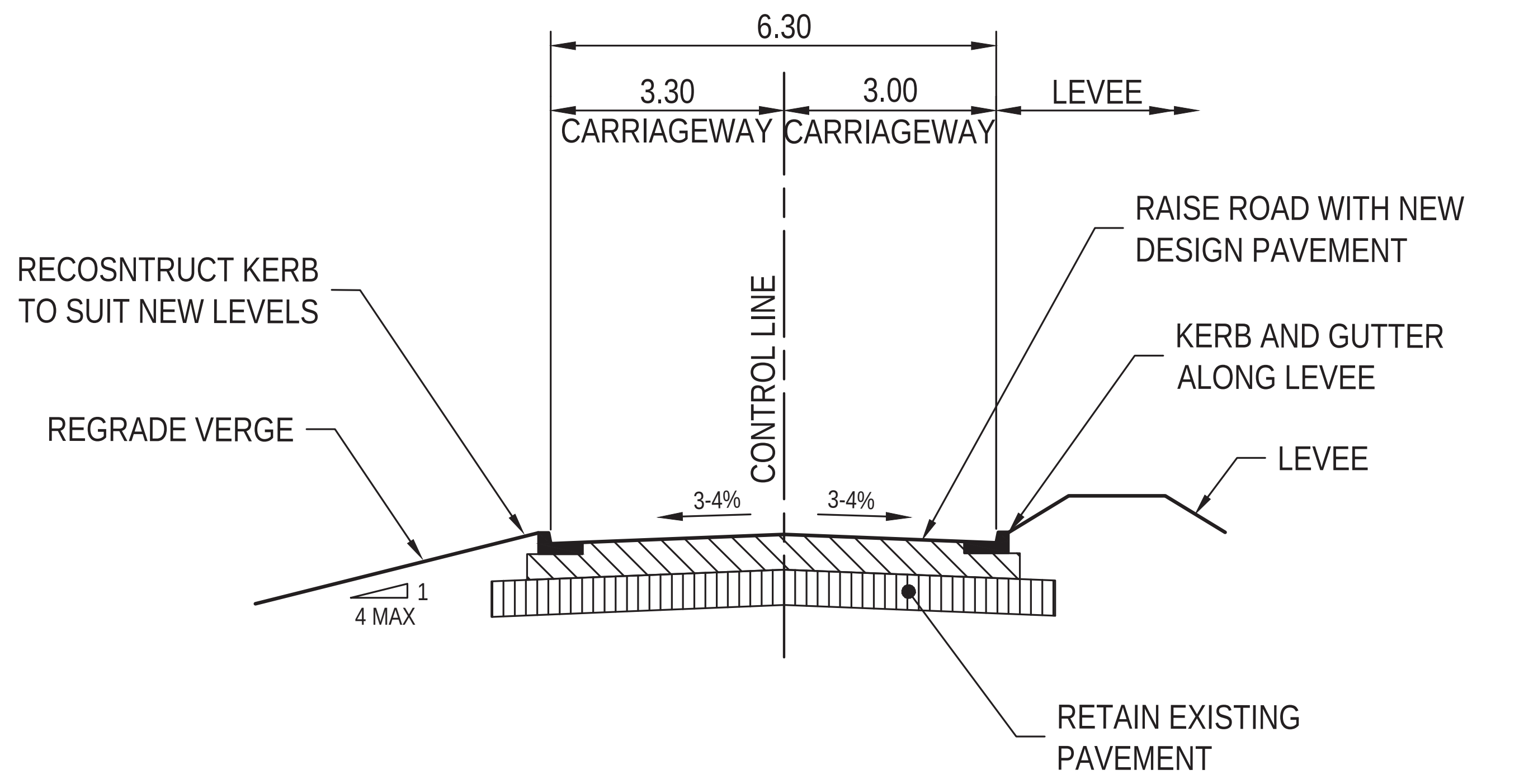

TYPICAL CROSS SECTION
LEONARD ROAD

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SECTION 4-4
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Project HANWOOD STORMWATER
PUMP AND LEVEE
CONCEPT DESIGN OPTIONS

Title CONCEPT INTERSECTION RAISING
TYPICAL CROSS SECTIONS

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APPENDIX

C

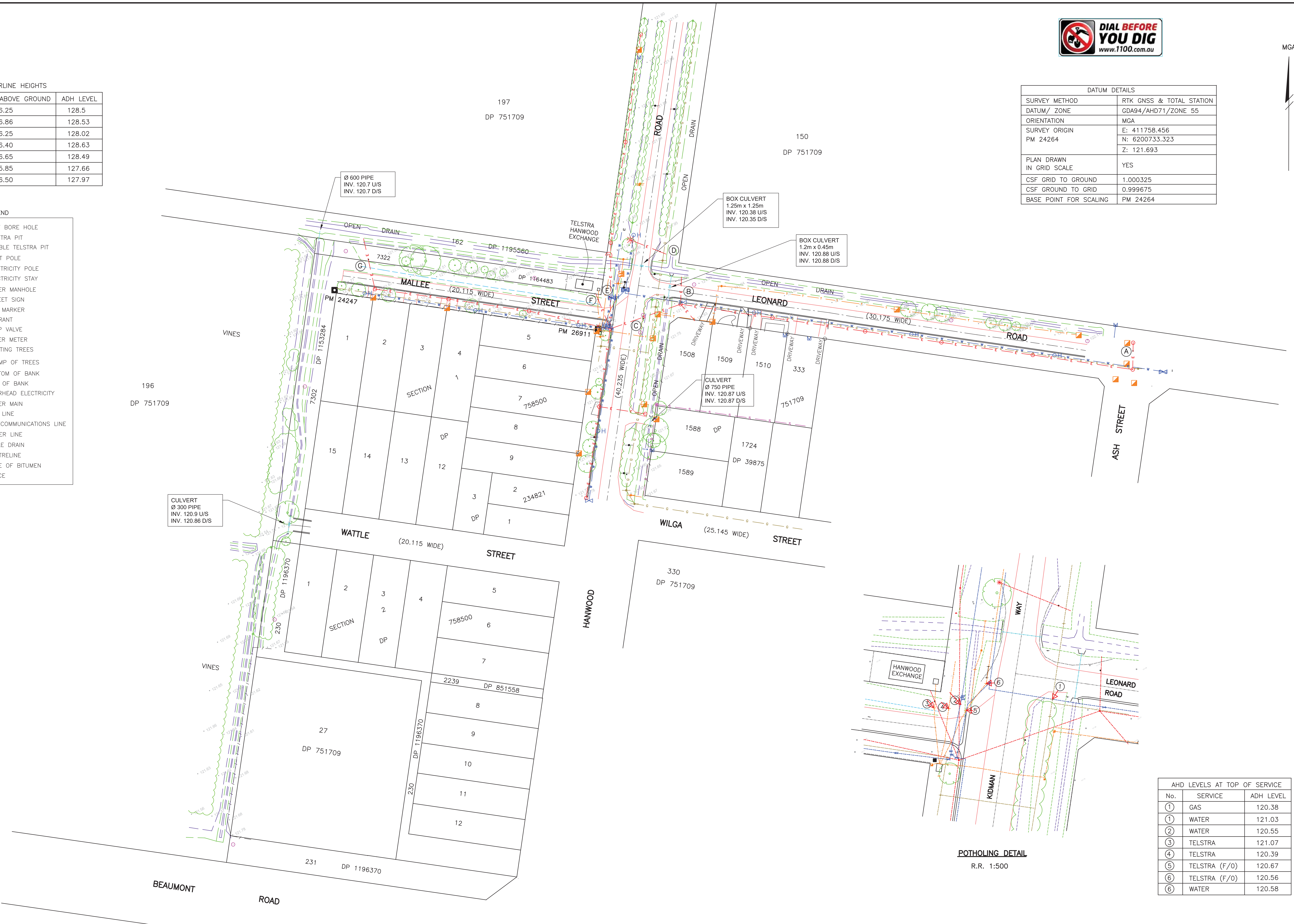
SURVEY/UTILITY POTHOLING

POWERLINE HEIGHTS

LOCATION	HEIGHT ABOVE GROUND	ADH LEVEL
Ⓐ	6.25	128.5
Ⓑ	6.86	128.53
Ⓒ	6.25	128.02
Ⓓ	6.40	128.63
Ⓔ	6.65	128.49
Ⓕ	5.85	127.66
Ⓖ	6.50	127.97

LEGEND

⊙	TEST BORE HOLE
◩	TELSTRA PIT
◩	DOUBLE TELSTRA PIT
✳	LIGHT POLE
—E —⊙E—	ELECTRICITY POLE
	ELECTRICITY STAY
⊛	SEWER MANHOLE
•—	STREET SIGN
	GAS MARKER
○H	HYDRANT
⋈	STOP VALVE
M	WATER METER
⊙	EXISTING TREES
	CLUMP OF TREES
— — —	BOTTOM OF BANK
— — —	TOP OF BANK
—— E ——	OVERHEAD ELECTRICITY
—— W ——	WATER MAIN
—— G ——	GAS LINE
—— T ——	TELECOMMUNICATIONS LINE
—— S ——	SEWER LINE
— — —	TABLE DRAIN
— · — · —	CENTRELINE
————	EDGE OF BITUMEN
—/—/—/—	FENCE



Ø 600 PIPE
INV. 120.7 U/S
INV. 120.7 D/S

BOX CULVERT
1.25m x 1.25m
INV. 120.38 U/S
INV. 120.35 D/S

BOX CULVERT
1.2m x 0.45m
INV. 120.88 U/S
INV. 120.88 D/S

CULVERT
Ø 750 PIPE
INV. 120.87 U/S
INV. 120.87 D/S

CULVERT
Ø 300 PIPE
INV. 120.9 U/S
INV. 120.86 D/S

POTHOLING DETAIL
R.R. 1:500

DATUM DETAILS

SURVEY METHOD	RTK GNSS & TOTAL STATION
DATUM/ ZONE	GDA94/AHD71/ZONE 55
ORIENTATION	MGA
SURVEY ORIGIN PM 24264	E: 411758.456
	N: 6200733.323
	Z: 121.693
PLAN DRAWN IN GRID SCALE	YES
CSF GRID TO GROUND	1.000325
CSF GROUND TO GRID	0.999675
BASE POINT FOR SCALING	PM 24264

AHD LEVELS AT TOP OF SERVICE

No.	SERVICE	ADH LEVEL
①	GAS	120.38
①	WATER	121.03
②	WATER	120.55
③	TELSTRA	121.07
④	TELSTRA	120.39
⑤	TELSTRA (F/0)	120.67
⑥	TELSTRA (F/0)	120.56
⑥	WATER	120.58

LIABILITY LIMITED BY A SCHEME APPROVED UNDER PROFESSIONAL STANDARDS LEGISLATION

ORIGINAL A1 SHEET

200mm

100mm

0

DATE	No.	ISSUE	AUTHORISED
01/05/2018	2A	POTHOLING ADDED	JH
18/04/2018	1A	ISSUED TO CLIENT	JH

DRAWN: JM CHECKED: JH

COMPUTER FILES: 13070a18_detail.dwg

COMMENTS:
BOUNDARIES NOT FULLY INVESTIGATED.
UNDERGROUND SERVICES NOT FULLY INVESTIGATED.
CONTACT RELEVANT AUTHORITIES PRIOR TO EXCAVATION.

phl surveyors

POLKINGHORNE HARRISON LONGHURST
115 Yambil Street, Griffith NSW 2680
13 Gibraltar Street, Bungendore NSW 2621
www.phlsurveyors.com.au

CLIENT: CARDNO
TITLE: PROPOSED HANWOOD TOWN LEVEE
DETAIL SURVEY NORTH AND WEST OF THE VILLAGE OF HANWOOD.

DATE: 12/04/2018
FILE: PN 13070
SCALE: 1:1000
A1– 524320_2A
SHEET 1
OF 1

Hanwood Stormwater Pump and Levee Design

APPENDIX

D

GEOTECHNICAL REPORT

Report on Geotechnical Investigation

Hanwood Stormwater Pump & Levee

80518062-002.0

Prepared for
Griffith City Council

4 August 2018

Contact Information

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Document Information

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Author(s):

Inglis Brien
Geotechnical Engineer

Effective Date 4/08/2018

Approved By:

Ian Piper
Principal Technical Officer

Date Approved 4/08/2018

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Table of Contents

Appendices

Tables

1 Introduction

1.1 Overview

This report presents the results of geotechnical investigation undertaken by Cardno (NSW/ACT) Pty Ltd for Griffith City Council (GCC) for the proposed Hanwood Stormwater Pump & Levee construction project. The project comprises works to mitigate flooding issues in the north western area of Hanwood in proximity to Kidman Way.

The proposed works comprise :

- Construction of a flood levee (embankment) along DC Handepot & DC DA;
- Installation of several one way valves within the levee adjacent the Mallee Street & Leonard Road;
- Installation of a pump station and associated infrastructure at Mallee Street; and
- Reconstruction of road pavement at intersection of Kidman Way, Mallee Street & Leonard Road.

The work was conducted at the request of Mr Brett Stonestreet on behalf of GCC on the 20 of March 2018 and was generally conducted in accordance with Cardno Fee Proposal reference 48980518.001.0385 dated 26 of February 2018.

This report should be read in conjunction with Cardno Civil Plans *“80518062-CI-100 Series Drawings”.*

1.2 Proposed Levee

Following the flooding of Hanwood in March 2012, Griffith City Council conducted a flood study & floodplain risk management study and plan which identified that the area required flood mitigation works, comprising construction of a flood levee and stormwater pumping station.

The purpose of the proposed pump and levee is to limit the extent of backwater flooding throughout Hanwood. The flooding is known to occur when Main Drain ‘J’ to the North of Hanwood, is at capacity which extends backwater along DC ‘A’ to the south. This is known to cause significant out of bank flooding throughout the Hanwood Area, which can last for numerous days until the tail water from Main Drain ‘J’ lowers. The proposed levee will aim to prevent backwater flooding effects on the village, with the stormwater pump discharging overland flows which accumulate on the Hanwood side of the levee.

1.3 Objectives

This geotechnical report has been prepared to assist in the detailed design and construction of the proposed levee and stormwater pump station. The report outlines the investigation findings and provides comments on the implication of the geotechnical conditions as well as design and construction implications comprising:

- Excavation conditions & excavatability of the subsurface profile;
- Foundation conditions, groundwater conditions and comment on any dewatering requirements;
- Geotechnical design parameters for all structural elements;
- Allowable soil bearing capacities;
- Pavement design for the embankment intersection with Kidman Way; and
- A general description of surface and subsurface conditions encountered.

2 Site Description

The levee is aligned along and within the northern road reserve of Leonard Road, and along the northern and western road reserve of Mallee Street as shown on Figure 1 attached as Appendix A. The proposed alignment intersects Kidman Way on the northern side of parallel running Mallee Street and Leonard Rd and adjacent to DC-DA. At the time of investigation, the surrounding drainage channels, DC-DA and DC-Handepot were dry.

The overall alignment is located within regionally low-lying terrain, with local topography characterised by flat alluvial flood plains associated with the Murrumbidgee River which is located approximately 25kms to the south. Vegetation across the site comprised predominantly light grass and scattered mature trees.

A high density of underground utilities (services) were noted during the investigation in the vicinity of the Kidman Way intersection. Services included fibre optics, natural gas, telecommunications, and potable water mains which were identified by various marker posts and review of the Dial Before You Dig (DBYD) plans. Service location was undertaken by a sub-consultant in conjunction with vacuum excavation during surveying.

3 Investigation Methodology

The fieldwork was undertaken on 30 April and 1 May 2018 and comprised excavation of eleven (11) test bores (BH01-BH11) along the alignment of the proposed levee and in the Kidman Way intersection area.

Site investigation was undertaken by an experienced Environmental / Geotechnical Scientist and comprised the following.

> A site walkover and visual inspection by an Environmental / Geotechnical Scientist from Cardno including site mapping and logging of significant site features.

> Underground utility location and surveying to inform both the geotechnical investigation scope & civil design.

> Excavation of three (3) test bores (BH01-BH03) using an 8 tonne tracked excavator fitted with a 300mm diameter auger, within proximity to proposed pavement reconstruction areas (shoulder of current travelling lanes of Kidman Way). The test bores were excavated to a target depth of approximately 1.2 m below ground level (BGL), with all test bores reaching the nominated target depth. Test bores were undertaken in the road shoulder due traffic management restrictions.

> Excavation of eight (8) test bores (BH04-BH011) using the same plant along the proposed levee alignment. The test bores were excavated to a target depth of 2.8 m BGL with all test bores reaching the nominated target depth.

> Dynamic Cone Penetrometer (DCP) tests were conducted adjacent to the test bores to aid in the assessment of subsurface strength conditions and consistencies.

> Disturbed bulk samples and environmental samples of natural materials were collected for subsequent laboratory testing.

Test bores were located by the use of a handheld GPS unit, as shown overlaid on aerial imagery on Figure 1 attached as Appendix A. Subsurface conditions are summarised in Section 4.2, and are detailed in the Engineering Logs attached in Appendix B along with explanatory notes.

4 Investigation Findings

4.1 Published Data

Reference to the Narrandera 1:250 000 Geological Sheet SI 55-10, Edition 2, 1977 [1] indicates that the site is located within Quaternary Age Flood Plain deposits (Qrs) of black and red clayey silt, sand and gravels.

Reference to the Australian Government Grape and Wine Research and Development Corporations (GWRDC) paper titled *'Soils of the Riverina'* [2] describes the site soils as sandy clay loam to light to heavy clay soils.

4.2 Subsurface Conditions

4.2.1 Levee Alignment

The subsurface profile encountered along the proposed levee alignment in boreholes (BH04-BH11) can be generally summarised as follows:

> **UNIT F - FILL**: Clayey GRAVE, Silty / Gravelly CLAY & CLAY encountered within six of the eight test bores (BH04 –BH05, BH07-BH09 & BH11) generally to depths of 0.1 m to 0.2 m BGL, with the exception of BH11 where fill was encountered up to 2.0m BGL. The materials were observed to be dry at the time of investigation, and of very stiff to hard / dense to very dense consistency. The materials are considered likely to comprise predominantly pavement materials and materials removed from the adjacent channel beds during maintenance; overlying

> **UNIT A – ALLUVIAL SOIL**: Predominantly medium to high plasticity CLAY encountered below the fill materials, with component of silt, sand and fine gravels (where present) to the depth of investigation in all locations. The material was observed to be dry of its plastic limit, and increasing in moisture content with depth increase BGL. The alluvial soils were assessed as very stiff to hard in consistency based on DCP testing undertaken.

4.2.2 Kidman Way Intersection

The subsurface conditions encountered in the three test bores drilled in the Kidman Way road shoulder pavement (BH01 – BH03) comprised:

> **WEARING COURSE**: Sprayed seal (multiple seals) to depths of 10-20 mm thickness; overlying

> **PAVEMENT:** Sandy GRAVEL pavement materials with component of silt and clay, to depths up to 0.3 m BGL. The pavement materials were observed to comprise fine to coarse, sub-rounded to angular gravels, of dry to moist condition at the time of investigation; overlying

> **SUBGRADE:** Silty / Sandy CLAY (similar to Unit A described above) at existing subgrade level to the limit of investigation. The material was observed to be dry of its plastic limit at the time of investigation, and was assessed as stiff to very stiff based on DCP testing undertaken.

4.2.3 General Comments

Details of the subsurface profiles encountered in the test bores are presented in the engineering logs attached in Appendix B together with explanatory notes.

Groundwater was not encountered during the investigation, however groundwater levels are likely to fluctuate with variations in climatic and site conditions. As noted the adjacent drainage channels were dry during the investigation, and groundwater may be present at the site when water is present within the channels.

4.3 Geotechnical Laboratory Test Results

Laboratory test results are summarised below, with complete laboratory test reports attached in Appendix C.

4.3.1 PSD & Atterberg Limits Testing

The results of Particle Size Distribution (PSD) and Atterberg limits testing undertaken on representative samples from Unit A are summarised below in Table 4-1.

Table 4-1 Summary of PSD & Atterberg testing Results

Location	Depth (m)	Material description	LL (%)	PL (%)	PI (%)	%Passing 2.36mm	%Passing 0.075mm
BH05	0.8-1.0	Silty CLAY, brown, trace sand & gravels	71	17	54	91	84
BH07	0.9-1.0	Silty CLAY, brown, trace sand	56	20	36	98	81
BH10	0.5-0.6	Silty CLAY; brown, trace sand	-	-	-	97	85

Notes to table:
LL – Liquid Limit
PL – Plastic Limit
PI – Plasticity Index

4.3.2 Material Density Test Results

The results of standard compaction tests undertaken on representative samples from Unit A are summarised below in Table 4-2.

Table 4-2 Summary of Material Density Test Results

Location	Depth (m)	Material Description	SOMC (%)	SMDD (t/m³)
BH06	0.7-1.0	Silty CLAY, brown	18.5	1.66
BH08	1.4-1.6	Silty CLAY, brown	18.5	1.72
BH11	2.0-2.2	Silty CLAY, brown	15.5	1.79

Notes to table:
SOMC – Standard Optimum Moisture Content
SMDD – Standard Maximum Dry Density

4.3.3 Shrink Swell Test Results

The results of the shrink swell tests undertaken on representative samples from Unit A are summarised below in Table 4-3 with the test report sheets attached in Appendix C.

Table 4-3 Summary of Shrink Swell Test Results

Location	Depth (m)	Sample Type	Soil Type	Swelling Strain (Esw %)	Shrinkage Strain (Esh %)	Shrink/Swell Index (Iss %)
BH04	1.0-1.2	U50	Silty CLAY; brown	0.0	2.1	1.2
BH09	0.9-1.0	U50	Silty CLAY; brown	1.8	2.6	2.0

Notes to table:
U50: Testing undertaken on thin walled 50mm diameter tube

4.3.4 Emerson Class Test Results

The results of an Emerson class test undertaken on a representative sample from Unit A is summarised below in Table 4-4.

Table 4-4 Summary of Emerson Class Test Results

Location	Depth (m)	Material Description	Emerson Class	Notes
BH007	0.4-0.5	Silty CLAY, brown	3	Dry soil does not disperse, however the soil remoulded at its plastic limit disperses.

4.3.5 California Bearing Ratio Test Results

The results of California Bearing Ratio (CBR) testing undertaken on representative samples from Unit A are summarised below in Table 4-5.

Table 4-5 Summary of Compaction and CBR Test Results

Location	Depth (m)	Material Description	W (%)	SOMC (%)	MDD (t/m^3)	Swell (%)	CBR (%)
BH001	0.5 – 0.8	Silty CLAY	21.9	21.0	1.64	1.0	8.0
BH002	0.5 – 0.8	Sandy CLAY	12.6	14.5	1.87	1.0	6.0

Notes:

W	Field Moisture Content
MDD	Maximum Dry Density (Standard compaction)
SOMC	Standard Optimum Moisture Content

4.3.6 Soil Salinity & Sodicity Test Results

Results of soil salinity and sodicity tests on representative samples from Unit A are summarised below in Table 4-6.

Table 4-6 Summary of Soil Salinity, Sodicity and Resistivity Test Results

Location	Depth (m)	pH	CEC (meq/100g)	ESP (%)
BH005	0.4-0.5	5.8	57	13.2
BH009	0.4-0.5	5.7	44	12.9

Notes to table:
meq/100g: milliequivalent per 100g of dry soil
CEC: Cation Exchange Capacity
ESP: Exchangeable Sodium Percentage

4.3.7 Soil Aggressivity Test Results

The results of the soil aggressivity test undertaken on representative samples from Unit A are summarised below in Table 4-7.

Table 4-7 Summary of Soil Aggressivity Test Results

Hole ID	Depth (m)	Soil Type and (Groundwater Condition)	pH(1:2) for concrete piles	EC (dS/cm)	Resistivity (Ωcm) Classification for steel	Sulphate (mg/kg), Classification for concrete	Chloride (mg/kg), Classification for concrete
BH004	0.4-0.5	Silty CLAY (B)	7.5	2.3	440	440	3300
BH008	0.4-0.5	Silty CLAY (B)	7.6	3.7	270	3500	1900

Notes to table:
- Exposure classification calculated in respect to both steel & concrete guidelines outlined in AS2159-2009.
- It should be noted that Resistivity is only relevant to exposed steel elements.
- Soil condition A will be encountered where structural elements are founded below the groundwater table, reference to site conditions and to the geotechnical logs should be made in order to determine the appropriate soil condition.

Non Aggressive
Mildly Aggressive
Moderately Aggressive
Severely Aggressive
Very Severely Aggressive
\- Not Tested/ Not Applicable

5 Proposed Levee Construction

5.1 Levee Construction

Levee construction is expected to comprise of an earthen embankment along the majority of the alignment. Where the proposed levee intersects Kidman Way it is expected that the entire road intersection (Kidman Way, Mallee St & Leonards Rd) would require modification of vertical alignments to match the proposed levee heights adjacent and as such the materials are expected to comprise of sealed pavement gravels in these locations. Pavement design can be seen detailed in Section 8 below.

All levee works, maintenance, planning and emergency works should be performed in accordance to the NSW Public Works for Justice NSW – Levee Owners Guideline [3]. Where discrepancies between this report and the guideline occur, consultation from an experienced civil / geotechnical engineer should be sought immediately.

5.2 Levee Design & Specification

The design and material specification for the proposed levee need to consider the following:

- Shrink swell related deformations resultant from seasonal moisture variations and fluctuations in moisture content;
- Deformations due to stress changes associated with water level fluctuations; and
- Consolidation of the foundation.

Table 5-1 below provides general material requirements and compaction specifications for the construction of the levee embankment.

Table 5-1 Levee Embankment Material Specification

Specifications	Zone 1 – Clay Core Material	Zone 2 – General Embankment Fill
Material Property		
Material Description	Sandy / Silty CLAYs with minor gravel content	
Shrink Swell Index (Iss %)	< 2%	
Plasticity Index	10-50%	
Permeability	< 10^{-9} m/s	N/A
Emerson Class	Minimum Class 4	Minimum Class 2
Maximum particle Size	50mm	75mm
Percentage Fine Content (Material Passing 0.075mm)	> 25%	> 20%
Compaction Requirements		
Compaction (Standard Relative Density AS1289 5.7.1)	Minimum 98%	Minimum 95%
Moisture Content	-1 to +2 of SOMC	-1 to +2 of SOMC

Notes to table:
SOMC: Standard Optimum Moisture Content
N/A: Not applicable

Considering the site geometry large volumes of fill required for the construction of the levee are not likely to be won from onsite sources and importation of offsite material is likely to occur. Some material may be generated during the construction of the pumping station, however considering the results of the laboratory testing undertaken on site soils, the subsurface clay material would likely require amelioration with gypsum or similar to render the material suitable for use as clay core material. Amelioration should be considered in conjunction with recommendations for the treatment and placement of highly saline soils as seen in Section 5.2.9.

Information provided by the Griffith City Council indicated that a potential borrow site for the import of material for the construction of the levee exists approximately 25kms to the south of Hanwood. Anecdotal evidence provided by the council indicated that the material has previously used in a flood levee on neighbouring council's project. Import material would be subject to adequate material testing and confirmation from an experienced geotechnical engineer.

Where geometry prevents the construction of a conventional earthen levee embankment, alternative retention systems may be implemented. Alternatives could comprise concrete barriers, continuous sheet piles or earthen embankments utilising soil-bentonite slurry walls / concrete cut off walls as an alternative to clay core.

5.2.2 Excavation Stability

Considering preliminary excavation depths of up to 5m to facilitate the construction of the pump station and the site investigation findings, excavations into the filling & underlying alluvial soils are expected to be readily undertaken using small to medium (3.5 to 15 tonne) excavation equipment.

Shallow excavations or trenches (less than 2.0 m depth) in the stiff or better alluvial clay soils would be expected to stand close to vertical in the short-term. Unsupported short-term excavations or trenches may undergo some local slumping into the excavation where elevated ground water conditions exist and seepage is encountered, this could occur after sustained periods of wet weather.

Where personnel are to enter excavations, options for short-term excavations include benching or battering back of the excavations at 2H:1V or the support of excavations within the alluvial clays or the adoption of suitable shoring systems such as trench boxes or slip form shoring.

5.2.3 Filling

Some minor site regrading is expected in the vicinity of the proposed levee to provide access to the levee and associate channels. Where general regrade (not including levee embankment fill) is undertaken it should be placed and compacted in accordance with AS 3798-2007 *Guidelines on Earthworks for Commercial and Residential Developments* [4].

Earthworks procedure should include the following:

- Removal of any existing uncontrolled filling, stockpiles, topsoil, slopewash / colluvium or deleterious materials from the areas where fill is to be placed. Any unsuitable material including foreign matter should be removed from the fill areas.
- Fill materials containing vegetation including tree stumps, roots, root fibres or other organic matter should be removed from site.
- Fill should not comprise material with particle sizes of greater than 100mm or 2/3 of the compacted layer thickness.
- Benching of the slopes where fill is to be placed with slopes steeper than 8H:1V will be required.
- Placement of fill in uniform horizontal layers with compaction of each layer to a minimum dry density ratio of 95% standard compaction (AS 1289-5.5.1) at moisture contents in the order of 85-115% of SOMC or ±2% but generally as close to SOMC as practical..

5.2.4 Embankment Batter Slopes

It should be noted that all batter slopes along the earthen levee alignment should be 1V:4H. Where batter slopes steeper than 1V:4H are proposed specific surface erosion control would need to be provided or a specific maintenance would be required. Surface erosion control could include vegetated jute mat or topsoiling of batter to encourage the development grasses and reduce erosion.

5.2.5 Vegetation

Large vegetation shall not be allowed to become established on or near the embankment. Tree roots (especially eucalyptus tree roots) can cause the core to crack and encourage piping development, resulting in the failure of the levee embankment.

All trees and shrubs shall be restricted to a minimum distance of 1.5 times the height of the tree away from the embankment, where restrictions occur consultation must be sought immediately.

5.2.6 Intersecting Services, Stormwater Outlets, One Way Flow Structures and Seepage Collars

Seepage collars will be required to be constructed along the discharge pipes traversing the levee embankment to increase the length of the percolation path and reduce the risk of piping developing around the pipes and resulting in failure of the levee.

Seepage collars are generally made of concrete with a required width depending on pipe diameter but are typically three times the pipe diameter.

5.2.7 Construction Monitoring

Variations in ground conditions are likely to occur between testing locations. There is the potential for soft alluvial soils of variable strength and uncontrolled filling to be encountered at variable depths along the proposed alignment. If conditions other than those described are encountered, further advice should be sought. During excavation, site inspections should be performed by an experienced geotechnical engineer to inspect founding conditions, excavation stability and other issues as discussed in this report.

5.2.8 Aggressivity of Site Material

With reference to AS2159-2009, the exposure classification for concrete elements founded in low permeability soils (Soil Condition B) such as silts and clays is non aggressive although concrete elements founded in ground water (Soil Conditions – A) would be mildly aggressive. Care should be taken during design as water table heights could vary significantly due to the site being located adjacent to a drainage channel.

5.2.9 Soil Salinity

Results of analytical testing of the soils at the Site were compared to the following guideline values derived from of *Department of Land Water Conservation NSW, 2002: Site Investigations for Urban Salinity* [5]. The adopted criteria based on the DLWC guidelines [5] are listed in Table 5-2 below.

Table 5-2 Salinity Class Assessment Criteria in Soil

Class	EC_e (dS/m)
Non- saline	<2
Slightly saline	2-4
Moderately saline	4-8
Very saline	8-16
Highly saline	>16

Based on the range of EC results indicated in the aggresivity testing undertaken on representative site soils in Table 4-7 and consideration of material types (heavy clay), a multiplication factor has been used for calculation of the EC_e. The conversion factor of 6 has been adopted based on the Table 6.1 of *Department of Land Water Conservation NSW, 2002: Site Investigations for Urban Salinity.*

Based on the summary of the laboratory results presented in Table 4-6 & 4-7 the site subsoils tested were observed to be very saline and highly saline.

Particular care must be taken to avoid the reversing or mixing the soil profile when cut and fill operations are undertaken, during construction of the pump station in particular where the effect will negatively impact on the salinity profile. The excavation and placement of in situ materials of high salinity could be coordinated with the excavation of non-saline to slightly saline imported material of similar consistency. This should have the effect of reducing overall salinity of the site soils. To minimise the impacts on the proposed structures forming the works, consideration should be given to:

- Minimising water infiltration;
- The use of native plants when landscaping;
- Retention of deep rooted vegetation;
- Minimising soil disturbance; and
- The use of higher strength concrete with thicker cover and exposure classifications or damp proof membranes.

5.2.10 Sodicity Assessment

Sodicity or exchangeable sodium percentage (ESP) is the measure of exchangeable sodium in the soil and relates to the likely dispersion on wetting and potential reactivity. In Australia, sodic soils are classified as soils with an ESP of 6-14% and highly sodic soils have an ESP of 15% or greater. On the basis of 2 samples tested for sodicity (ESP), testing indicates that the underlying alluvial soils are generally sodic.

Dispersion and erosion can be controlled by prompt replacement of topsoil up to 300 mm thick and revegetation on the area following construction. Gypsum treatment of clays used in the construction of the levee embankment where site won soils are utilised would likely be required.

6 Stormwater Pump Foundation Recommendations

Foundation for the stormwater pump station are likely to comprise of shallow foundations and as such design should be undertaken in accordance to relevant engineering standards and engineering principles. Foundation conditions are likely to comprise stiff to very stiff alluvial clay materials, as defined above as Unit A.

Laboratory shrink swell test results indicate that the tested clay soils are generally moderately reactive and as such design of the foundation system should consider the effects of shrink swell movements in accordance with AS2870-2011 [6]. Expected characteristic surface movements are in the order of 50-60mm.

6.1 Footings

All footings should be founded below any topsoil, uncontrolled fill or deleterious materials. All footings for the same structure should be founded on strata of similar stiffness and reactivity to minimise the risk of differential movements.

All footings excavations should be inspected prior to installation of structural steel by a suitably experienced engineer or geotechnical consultant to confirm that the founding conditions are as described in this report. All loose material should be cleared from the footing excavations before concrete is poured.

6.1.1 Shallow / High Level Foundations

Footings designed in accordance with engineering principles and founded in stiff or better soils (below topsoil, uncontrolled fill or other deleterious material) may be proportioned on an allowable bearing capacity of 150kPa. The founding conditions should be assessed by a geotechnical consultant or experienced engineer to confirm suitable conditions.

7 Preliminary Pavement Thickness Design

7.1 Proposed Works

The proposed levee alignment intersects Kidman Way near the intersection of Leonard Road & Kidman Way, and as such increases to the vertical alignment of Kidman Way and the adjoining Leonard Road & Mallee Street are required. Civil design available at the time of report preparation indicates an increase of up to 0.7 m is proposed in the intersection to achieve design levels and facilitate the required levee freeboard levels.

The extent of pavement works is presented on the site plan Figure 1 attached as Appendix A, and comprises approximately 200 m of the Kidman Way, and 30-35 m of Leonard Road and Mallee Street to accommodate the increase in design level and suitable grades back to existing road levels.

7.2 Supplied Data

The Kidman Way is a Roads and Maritime (RMS) asset, and the RMS have provided details of an intersection upgrade project located to the south of Hanwood (outside of the site) for consideration.

Information provided by RMS relevant to the proposed pavement works for the Hanwood Levee is summarised below.

> Existing Kidman Way pavement profile generally comprises a seal or thin asphalt of 20 mm to 50 mm thickness, overlying an unbound sandy GRAVEL of 200 mm to 230 mm thickness, overlying a variable clayey sand to sandy clay fill of approximately 100 mm thickness overlying a natural clay subgrade.

> A design subgrade CBR of 3% for the natural subgrade was adopted based on previous intrusive investigation and laboratory testing conducted.

> Design traffic adopted ranged from 1.5×10^7 Design Equivalent Standard Axles (DESA) for a 20 year design period, to 4.7×10^7 for 40 years.

> RMS are considering the option of in-situ stabilising the existing pavement materials to a depth of 220 mm (in conjunction with addition of 50 mm thickness of DGS20 material) and overlaying with a deep lift asphalt of 200 mm thickness.

7.3 Pavement Options

Several pavement options would be available for the proposed works, including:

> Flexible pavement, constructed from unbound flexible materials;

> Heavily bound pavement, constructed from imported heavily bound basecourse material (subject to material availability);

> Bound pavement, formed from in-situ stabilisation of the existing pavement materials; and

> Full depth asphalt pavement.

The following considerations have been made in regards to selecting a suitable pavement type for the works.

> The existing pavement composition is understood to be constructed from unbound granular materials, as indicated by the limited investigation and RMS supplied data, and matching of any new pavements would be beneficial.

> It is unknown if a local material source for plant mixed heavily bound material is available (and considered unlikely).

> Existing design levels are proposed to be raised significantly.

Based on the above factors, construction of a pavement utilising flexible materials is considered an appropriate option. This would be subject to RMS approval, and where alternate options are required amendments to this report would be required.

Given the increase in design level and design thicknesses presented in Section 7.4.3 below, overlay of the existing Kidman Way pavement would be feasible within approximately Chainage (Ch) 35 m to 165 m, with excavation to accommodate the minimum pavement thickness in the remaining sections.

Considering the relatively short sections of adjoining Leonard Road & Mallee Street pavement within the proposed works area and pavement tie in requirements, ease of construction etc. separate pavement designs have not been provided and it is assumed the sections would be constructed to RMS requirements. Review of the vertical alignment suggests excavation to accommodate the proposed pavement would generally be required (i.e. pavement overlay not feasible).

7.4 Flexible Pavement Option

It is understood RMS has not yet endorsed Austroads AGPT02-17, and as such pavement design presented below has been conducted in accordance with Austroads AGPT02-12 *Guide to Pavement Technology, Part 2: Pavement Structural Design* [7].

7.4.1 Design Traffic

Table 7-1 Design Traffic Adopted

Traffic Parameter	20 Year Design Period	40 Year Design Period
DESA	1.5×10^7	4.7×10^7
$DSAR_s$	2.4×10^7	7.5×10^7

Notes to table:
DESA: Design Equivalent Standard Axles
$DSAR_s$: Design number of Standard Axle Repetitions for rutting and loss of shape (subgrade strain)

A separate pavement design has not been provided for Leonard Road & Mallee Street, and where alternate design traffic is available for the sections, and subject to RMS approval, amendments to the designs presented herein could be considered.

7.4.2 Subgrade Conditions

The investigation conducted indicates soaked CBR values in the range of 6% to 8% for samples of the natural clay subgrade tested.

DCP test results obtained during the investigation suggest in-situ CBR values of >5% for the clay subgrade materials, with reference to Figure 5.3 from Austroads AGPT02-12 [7].

A design subgrade CBR of 5% is therefore considered appropriate and has been adopted for the pavement thickness design presented below.

It should be noted that the investigation was limited to the existing Kidman Way shoulders due to traffic control restrictions, and further investigation and confirmatory laboratory testing would be recommended over the works area as variation in the existing traveling lane pavements may change recommended designs.

7.4.3 Pavement Thickness Design

Pavement thickness design has been undertaken for the widening in accordance with the mechanistic procedure indicated in Austroads AGPT02-12 [7]. The software package CIRCLY 6.0 has been used to confirm the proposed pavement design, with CIRCLY output sheets included in Appendix D.

A suitable pavement design is shown below in Table 7-2 taking into consideration the adopted design traffic and subgrade conditions.

It should be noted that the layer thicknesses detailed in the following sections are minimum thicknesses regardless of construction tolerances.

Table 7-2 Unbound Granular Pavement Design

Layer	Thickness	
Wearing Course	Two coat sprayed seal (1)	
Basecourse (2)	180 mm	190 mm
Subbase	340 mm	380 mm
Select	- (3)	- (3)
Subgrade	Subgrade min. CBR 5%	
Minimum Total Thickness	520 mm	570 mm
Design Period	20 Years	40 Years
Design Traffic	1.5×10^7 DESA	4.7×10^7 DESA

Notes to table:
(1) Sprayed seal surface recommended to match existing seal type.
(2) Minimum basecourse thickness required for design traffic based on Figure 8.4 from Austroads AGPT02-12 [7].
(3) Select layer could be considered and may be beneficial in overlay areas, subject to RMS approval. Design amendments would be required where minimum thicknesses above are proposed to be altered.

The minimum total thickness presented above is the minimum cover to subgrade required, and the difference between proposed design level and base of existing pavement (from available information) would exceed the minimum total thickness within approximately Ch 35-165 m. This would provide the capacity to overlay the existing pavement within this area, which is identified on Figure 2 attached in Appendix A as ‘Approximate extent of pavement overlay’.

The overlay should comprise construction of the basecourse and subbase layer thicknesses presented in Table 7-2 above at a minimum, and either thickening of the subbase or addition of a select layer where required (i.e. in the middle of the Kidman Way section where the height increase is the deeper). Additional boxing at the tie in locations would be required to accommodate the minimum total thickness, within approximately Ch 0-35 m and Ch 165-202 m. Where existing pavement materials are generated in these areas the materials may be suitable for reuse as subbase or select material, subject to confirmatory testing to confirm RMS 3051 [8] or RMS 3071 [9] requirements are achieved.

Potential impacts of flooding to the pavement requires careful consideration with adoption of the above flexible pavement design. Particular care is required to provide a waterproof seal, along with adequate drainage (discussed further in the following report sections). Impacts to the pavement would be dependent on the length of inundation, and where the pavement is unable to be adequately protected, stabilisation to form a bound pavement would be required.

7.4.4 Subgrade Preparation

Subgrade preparation for pavement areas should be in general accordance with RMS QA Specification R44 [10].

Recommended treatment for the existing subgrade or select subgrade in construction of a flexible pavement is as follows:

- Possible removal of the seal (if required) for offsite disposal or recycling within pavement overlay areas;
- Excavation to design subgrade level outside of pavement overlay areas, with the stockpiling of the existing pavement material for reuse upon approval. Where the works are required to be undertaken in accordance with RMS R44 specification [10], deeper excavation would be required to achieve the minimum 1.2 m depth requirement within transition zone areas;
- Following excavation to design subgrade level, the existing pavement would be exposed in the majority of areas, however medium to high plasticity clay soils are expected to be exposed in some areas. It should be noted that in periods of heavy rainfall, the clays could some construction difficulties in trafficability and as such, allowances should be made for appropriate techniques and construction plant;

- Given the proposed vertical alignment and RMS R44 requirements [10], filling to achieve design levels is expected to comprise predominantly select quality material, which should comply with RMS 3071 requirements [9];
- Proof rolling of the exposed subgrade with a heavy (minimum 10 tonne static) roller. Loose or yielding areas detected during the proof rolling should be excavated and replaced with compacted select fill or subgrade replacement;
- Select material placed should be compacted to at least 100% of standard maximum dry density (refer Table 7-3 below for full details).

Following satisfactory preparation of the subgrade, the pavement should be placed in accordance with the requirements of the appropriate section of this report and RMS Specifications.

7.4.5 Pavement Interface

Where the new pavement abuts existing sections, care shall be taken to bench the basecourse layers into the existing pavement, in combination with resealing over the interface to prevent moisture penetration into the existing / new pavement.

It should be noted that when variable pavements are abutted the potential for localised failure is greater. Care should be exercised in the placement and compaction of the subgrade and pavements in this area to maximise the performance of the pavement. Intra-pavement drainage should also be installed at subgrade level at the interfaces of existing and new pavement sections (refer Section 7.4.6 below).

Consideration should also be given to sealing any cracks that may develop between existing and new pavements, benching to tie in pavements and the use of a strain relieving membranes at the interface may be appropriate.

7.4.6 Pavement Drainage

The moisture regime associated with a pavement has a major influence on the performance of the pavement since the stiffness/strength of the pavement materials is dependent on the moisture content of the materials. The site is located in an area subject to inundation during flooding, and particular care is required to provide a waterproof seal for the pavement materials, along with adequate surface and sub-surface drainage to ensure the unbound granular materials do not become wet and loose stiffness/strength.

An intra-pavement drain should be provided at the interface between any sections of variable pavements, and where new pavements join to existing pavements. Intra-pavement subsoil drains should be in accordance with RMS QA Specification R37 [11] or equivalent and should penetrate to the subgrade or to the base of any replaced subgrade material.

Attention to detail in drainage design and construction is essential for optimum performance. Expensive drainage systems can be blocked or otherwise prevented from operating by inappropriate construction procedures or drainage design. Poor performance of a drainage system can, in turn, result in major deficiencies in the pavement performance. The selection, construction and maintenance of appropriate drainage mechanisms and construction materials that are durable and insensitive to moisture change is essential.

7.4.7 Pavement Materials

Pavement materials and compaction requirements should conform to those outlined below.

Table 7-3 Pavement Materials and Compaction Requirements

Pavement Course	Material Specification	Compaction Requirements
Basecourse High quality crushed rock base material	Complying with RMS QA Specification 3051 Category A [8] and CBR > 120%, 2% < PI < 6%	Min 100% Modified (RMS T112) (60-90% of SOMC)
Subbase Quality crushed rock subbase material	Complying with RMS QA Specification 3051 Category A [8] and CBR > 20%, PI < 6%	Min 102% Standard (RMS T111) (60-90% of SOMC)
Selected Material	Material complying with RMS QA Spec 3071 and CBR > 30%, PI < 15%	Min 95% Modified (RMS T112) or 100% Standard (RMS T111)

Minimum testing on all potential pavement select materials should include 10 day soaked CBR, Atterberg Limits and Particle Size Distribution analysis. Pre-treatment of materials prior to testing would be advisable for material subject to breakdown.

Wearing Courses should be designed using either RMS Sprayed Sealing Guide [12], QA Specifications R106 [13] and R111 [14] or RMS QA Specification R116 [15] using Austroads APRG Report No. 18 [16] methodology.

7.4.8 Construction Materials and References

All works and materials used in the construction of the pavement should comply with RMS specifications and those outlined indicated in this report. Where discrepancies may occur clarification should be sought from the RMS on their requirements. Material should be selected to be compatible with the design and the existing pavement material.

As mentioned the site is subject to periodic inundation, and the use of low permeability materials in the verges could assist with limiting moisture ingress into the pavement materials via the shoulder.

It is suggested that the pavement designer be consulted prior to the use of alternate materials. Contractors should specify materials to be used in construction at the time of tendering, with all materials to be approved by the client prior to incorporation in the works.

7.4.9 Construction Inspections

The subgrade and existing pavement will require inspection by an experienced geotechnical consultant after boxing out and prior to filling to design subgrade level. The purpose of inspections is to confirm design parameters, assess the suitability of the subgrade to support the pavement, and delineate areas which may require subgrade replacement or remedial treatment prior to construction.

8 Limitations

Cardno has performed investigation and consulting services for this project in general accordance with current professional and industry standards. The extent of testing was limited to discrete test locations and variations in ground conditions can occur between test locations that cannot be inferred or predicted.

A geotechnical consultant or qualified engineer shall provide inspections during construction to confirm assumed conditions in this assessment. If subsurface conditions encountered during construction differ from those given in this report, further advice shall be sought without delay.

Cardno, or any other reputable consultant, cannot provide unqualified warranties nor does it assume any liability for the site conditions not observed or accessible during the investigations. Site conditions may also change subsequent to the investigations and assessment due to ongoing use.

This report and associated documentation was undertaken for the specific purpose described in the report and shall not be relied on for other purposes. This report was prepared solely for the use by Griffith City Council and any reliance assumed by other parties on this report shall be at such parties own risk.

10 References

[1] Narrandera Geology Map, “Narrandera 1;250 000 Gelogical Sheet SI 55-10, 2nd Edition,” Geological Survey of New South Wales, 1977.

[2] Australian Governement - Grape & Wine Research Development Corporation , Soils of the Riverina, Austrilan Government , September 2012.

[3] NSW Public Works for Justice NSW, “Levee Owners Guide,” NSW Government , September 2015.

[4] Australian Standard AS3798-2007, “Guidelines on Earthworks for Commercial and Residential Structures,” Standards Australia, 2007.

[5] Department of Land and Water Conservation, Site investigations for Urban Salinity, Department of Land and Water Conservation NSW, 2002.

[6] Australian Standard AS2870-2011, “Residential Slabs and Footings,” Standards Australia, 2011.

[7] Austroads AGPT02-12, “Guide to Pavement Technology Part 2: Pavement Structural Design,” Austroads Ltd, 2012.

[8] RMS QA Specification 3051 (Ed 6 Rev 5), “Granular Base and Subbase Materials for Surfaced Road Pavements,” Roads and Maritime Services, November 2014.

[9] RMS QA Specification 3071 (Edition 2 / Revision 1), “Selected Material for Formation,” Roads and Maritime Services, May 2017.

[10] RMS QA Specification R44 (Ed 3 Rev 15), “Earthworks,” Roads and Maritime Services, October 2011.

[11] RMS QA Specification R37 (Ed 4 Rev 1), “Intra-pavement Drains”.

[12] RMS TP-GLD-001 (Ed 2), “Sprayed Sealing Guide”.

[13] RMS QA Specification R106 (Ed 4 Rev 0), “Sprayed Bituminous Surfacing (with Cutback Bitumen)”.

[14] RMS QA Specification R111 (Ed 2 Rev 0), “Spayed Bituminous Surfacing (with Bitumen Emulsion)”.

[15] RMS QA Specification R116 (Ed 8 Rev 2), “Heavy Duty Dense Draded Asphalt”.

[16] Austroads APRG Report No. 18, “Selection & design of asphalt mixes: Australian provisional guide”.

APPENDIX

A

DRAWINGS

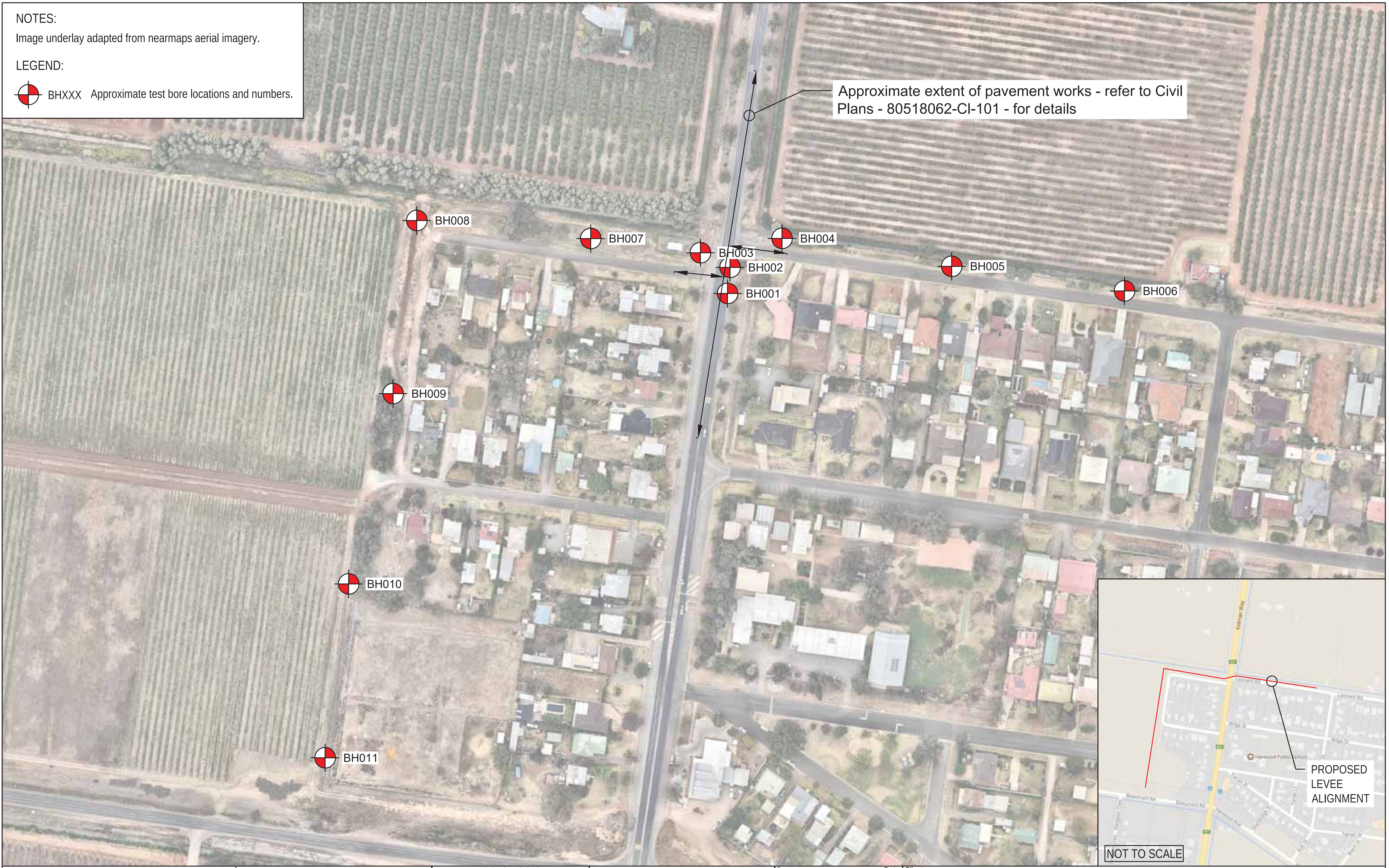
DATE PLOTTED: 23 July 2018 5:24 PM BY : ABID ALI

NOTES:

Image underlay adapted from nearmaps aerial imagery.

LEGEND:

⊕ BHXXX Approximate test bore locations and numbers.



0 50m

SCALE 1:2000 @A3

NORTH

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Cardno
Shaping the Future

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Web: www.cardno.com.au

Drawn	AA	Date 23/07/2018
Checked	DGB	Date 23/07/2018
Designed		Date
Verified		Date
Approved		

Client
Griffith City Council

Project
Hanwood Stormwater Pump & Levee

Title
Testing Location Plan

Status
FOR INFORMATION ONLY
NOT TO BE USED FOR CONSTRUCTION PURPOSES

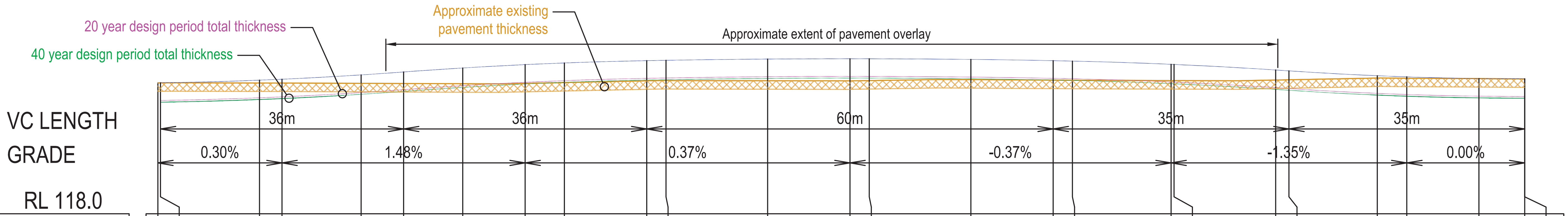
Project Number	Scale	Size
80518062	1:2000	A3

Figure Number	Revision
Figure 1	A

XREF's:
CAD File: N:\Projects\805\FY18\062_HANWOOD LEVEE V2\Drawings\Build\Geotechnical\Site Investigation Plan.dwg

=

DATE PLOTTED: 25 July 2018 8:19 AM BY : DAVE BASTIAN



CUT/FILL DEPTH	-0.000	0.000	0.083	0.113	0.247	0.341	0.463	0.551	0.576	0.594	0.594	0.638	0.652
DESIGN SURFACE	121.859	121.861	121.940	121.967	122.094	122.180	122.298	122.397	122.445	122.514	122.524	122.561	122.570
NATURAL SURFACE	121.859	121.860	121.856	121.854	121.847	121.840	121.835	121.846	121.870	121.920	121.930	121.923	121.918
CHAINAGE	0.000	0.379	15.000	18.323	30.000	36.268	45.000	54.212	60.000	72.156	75.000	90.000	102.156

CUT/FILL DEPTH	0.641	0.596	0.574	0.565	0.479	0.475	0.296	0.263	0.068	0.039	0.001	-0.000	-0.000
DESIGN SURFACE	122.569	122.550	122.514	122.502	122.406	122.402	122.239	122.213	122.070	122.037	121.987	121.978	121.978
NATURAL SURFACE	121.928	121.954	121.940	121.937	121.927	121.927	121.943	121.951	122.002	121.998	121.986	121.978	121.978
CHAINAGE	105.000	120.000	132.156	135.000	149.556	150.000	165.000	166.956	180.000	184.355	195.000	201.755	201.762

PAVEMENT OVERLAY EXTENT

V 1:125

0 2.5 5m

SCALE 1:125 VERTICAL @A3

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Drawn DGB Date 25.07.2018
Checked IB Date 25.07.2018
Designed Date
Verified Date
Approved

Client Griffith City Council
Project Hanwood Stormwater Pump & Levee
Title Long Section Plan with Pavement Overlay Extent

Status FOR INFORMATION ONLY
NOT TO BE USED FOR CONSTRUCTION PURPOSES
Project Number 80518062
Scale V 1:125
Size A3
Figure Number Figure 2
Revision A

XREF's:
CAD File: N:\Projects\805\FY18\062_HANWOOD LEVEE V2\Design and Analysis\Geotech\Drawings\dwg files\Geotech Site Plan 80518062-Geo.dwg
=

APPENDIX

B

ENGINEERING LOGS

Explanatory Notes

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS1726-2017 Geotechnical Site Investigations. Material descriptions are deduced from field observation or engineering examination, and may be appended or confirmed by in situ or laboratory testing. The information is dependent on the scope of investigation, the extent of sampling and testing, and the inherent variability of the conditions encountered.

Subsurface investigation may be conducted by one or a combination of the following methods.

Method	
Test Pitting: excavation/trench	
BH	Backhoe bucket
EX	Excavator bucket
R	Ripper
H	Hydraulic Hammer
X	Existing excavation
N	Natural exposure
Manual drilling: hand operated tools	
HA	Hand Auger
Continuous sample drilling	
PT	Push tube
PS	Percussion sampling
SON	Sonic drilling
Hammer drilling	
AH	Air hammer
AT	Air track
Spiral flight auger drilling	
AS	Auger screwing
AD/V	Continuous flight auger: V-bit
AD/T	Continuous spiral flight auger: TC-Bit
HFA	Continuous hollow flight auger
Rotary non-core drilling	
WB	Washbore drilling
RR	Rock roller
Rotary core drilling	
PQ	85mm core (wire line core barrel)
HQ	63.5mm core (wire line core barrel)
NMLC	51.94mm core (conventional core barrel)
NQ	47.6mm core (wire line core barrel)
DT	Diatube (concrete coring)

Sampling is conducted to facilitate further assessment of selected materials encountered.

Sampling method	
Soil sampling	
B	Bulk disturbed sample
D	Disturbed sample
C	Core sample
ES	Environmental soil sample
SPT	Standard Penetration Test sample
U	Thin wall tube ‘undisturbed’ sample
Water sampling	
WS	Environmental water sample

Field testing may be conducted as a means of assessment of the in situ conditions of materials.

Field testing		
SPT	Standard Penetration Test	
HP/PP	Hand/Pocket Penetrometer	
Dynamic Penetrometers (blows per noted increment)		
	DCP	Dynamic Cone Penetrometer
	PSP	Perth Sand Penetrometer
MC	Moisture Content	
VS	Vane Shear	
PBT	Plate Bearing Test	
IMP	Borehole Impression Test	
PID	Photo Ionization Detector	

If encountered, refusal (R), virtual refusal (VR) or hammer bouncing (HB) of penetrometers may be noted.

The quality of the rock can be assessed by the degree of natural defects/fractures and the following.

Rock quality description	
TCR	Total Core Recovery (%) (length of core recovered divided by the length of core run)
RQD	Rock Quality Designation (%) (sum of axial lengths of core greater than 100mm long divided by the length of core run)

Notes on groundwater conditions encountered may include.

Groundwater	
Not Encountered	Excavation is dry in the short term
Not Observed	Water level observation not possible
Seepage	Water seeping into hole
Inflow	Water flowing/flooding into hole

Perched groundwater may result in a misleading indication of the depth to the true water table. Groundwater levels are also likely to fluctuate with variations in climatic and site conditions.

Notes on the stability of excavations may include.

Excavation conditions	
Stable	No obvious/gross short term instability noted
Spalling	Material falling into excavation (minor/major)
Unstable	Collapse of the majority, or one or more face of the excavation

Explanatory Notes: General Soil Description

The methods of description and classification of soils used in this report are based on Australian Standard AS1726-2017 Geotechnical Site Investigations. In practice, a material is described as a soil if it can be remoulded by hand in its field condition or in water. The dominant component is shown in upper case, with secondary components in lower case. In general descriptions cover: soil type, plasticity or particle size/shape, colour, strength or density, moisture and inclusions.

In general, soil types are classified according to the dominant particle on the basis of the following particle sizes.

Soil Classification		**Particle Size (mm)**
CLAY		< 0.002
SILT		0.002 0.075
SAND	fine	0.075 to 0.21
	medium	0.21 to 0.6
	coarse	0.6 to 2.36
GRAVEL	fine	2.36 to 6.7
	medium	6.7 to 19
	coarse	19 to 63
COBBLES		63 to 200
BOULDERS		> 200

Soil types may be qualified by the presence of minor components on the basis of field examination methods and/or the soil grading.

Terminology	**In coarse grained soils**		**In fine soils**
	% fines	**% coarse**	**% coarse**
Trace	≤5	≤15	≤15
With	>5, ≤12	>15, ≤30	>15, ≤30

The strength of cohesive soils is classified by engineering assessment or field/lab testing as follows.

Strength	**Symbol**	**Undrained shear strength**
Very Soft	VS	≤12kPa
Soft	S	12kPa to ≤25kPa
Firm	F	25kPa to ≤50kPa
Stiff	St	50kPa to ≤100kPa
Very Stiff	VSt	100kPa to ≤200kPa
Hard	H	>200kPa

Cohesionless soils are classified on the basis of relative density as follows.

Relative Density	**Symbol**	**Density Index**
Very Loose	VL	<15%
Loose	L	15% to ≤35%
Medium Dense	MD	35% to ≤65%
Dense	D	65% to ≤85%
Very Dense	VD	>85%

The plasticity of cohesive soils is defined by the Liquid Limit (LL) as follows.

Plasticity	**Silt LL**	**Clay LL**
Low plasticity	≤ 35%	≤ 35%
Medium plasticity	N/A	> 35% ≤ 50%
High plasticity	> 50%	> 50%

The moisture condition of soil (*w*) is described by appearance and feel and may be described in relation to the Plastic Limit (PL), Liquid Limit (LL) or Optimum Moisture Content (OMC).

Moisture condition and description	
Dry	Cohesive soils: hard, friable, dry of plastic limit. Granular soils: cohesionless and free-running
Moist	Cool feel and darkened colour: Cohesive soils can be moulded. Granular soils tend to cohere
Wet	Cool feel and darkened colour: Cohesive soils usually weakened and free water forms when handling. Granular soils tend to cohere

The structure of the soil may be described as follows.

Zoning	**Description**
Layer	Continuous across exposure or sample
Lens	Discontinuous layer (lenticular shape)
Pocket	Irregular inclusion of different material

The structure of soil layers may include: defects such as softened zones, fissures, cracks, joints and root-holes; and coarse grained soils may be described as strongly or weakly cemented.

The soil origin may also be noted if possible to deduce.

Soil origin and description	
Fill	Anthropogenic deposits or disturbed material
Topsoil	Zone of soil affected by roots and root fibres
Peat	Significantly organic soils
Colluvial	Transported down slopes by gravity/water
Aeolian	Transported and deposited by wind
Alluvial	Deposited by rivers
Estuarine	Deposited in coastal estuaries
Lacustrine	Deposited in freshwater lakes
Marine	Deposits in marine environments
Residual soil	Soil formed by in situ weathering of rock, with no structure/fabric of parent rock evident
Extremely weathered material	Formed by in situ weathering of geological formations, with the structure/fabric of parent rock intact but with soil strength properties

The origin of the soil generally cannot be deduced solely on the appearance of the material and the inference may be supplemented by further geological evidence or other field observation. Where there is doubt, the terms ‘possibly’ or ‘probably’ may be used

Explanatory Notes: General Rock Description

The methods of description and classification of rocks used in this report are based on Australian Standard AS1726-2017 Geotechnical Site Investigations. In practice, if a material cannot be remoulded by hand in its field condition or in water, it is described as a rock. In general, descriptions cover: rock type, grain size, structure, colour, degree of weathering, strength, minor components or inclusions, and where applicable, the defect types, shape, roughness and coating/infill.

Rock types are generally described according to the predominant grain or crystal size, and in groups for each rock type as follows.

Rock type	Groups
Sedimentary	Deposited, carbonate (porous or non), volcanic ejection
Igneous	Felsic (much quartz, pale), Intermediate, or mafic (little quartz, dark)
Metamorphic	Foliated or non-foliated
Duricrust	Cementing minerology (iron oxides or hydroxides, silica, calcium carbonate, gypsum)

Reference should be made to AS1726 for details of the rock types and methods of classification.

The classification of rock weathering is described based on definitions in AS1726 and summarised as follows.

Term and symbol		Definition
Residual Soil	RS	Soil developed on rock with the mass structure and substance of the parent rock no longer evident
Extremely weathered	XW	Weathered to such an extent that the rock has ‘soil-like’ properties. Mass structure and substance still evident
Distinctly weathered	DW	The strength is usually changed and may be highly discoloured. Porosity may be increased by leaching, or decreased due to deposition in pores. May be distinguished into MW (Moderately Weathered) and HW (Highly Weathered).
Slightly weathered	SW	Slightly discoloured; little or no change of strength from fresh rock
Fresh Rock	FR	The rock shows no sign of decomposition or staining

The rock material strength can be defined based on the point load index as follows.

Term and symbol		Point Load Index $I_{s}50$ (MPa)
Very Low	VL	0.03 to 0.1
Low	L	0.1 to 0.3
Medium	M	0.3 to 1.0
High	H	1.0 to 3
Very High	VH	3 to 10
Extremely High	EH	> 10

It is important to note that the rock material strength as above is distinct from the rock mass strength which can be significantly weaker due to the effect of defects.

A preliminary assessment of rock strength may be made using the field guide detailed in AS1726, and this is conducted in the absence of point load testing.

The defect spacing measured normal to defects of the same set or bedding, is described as follows.

Definition	Defect Spacing (mm)
Thinly laminated	< 6
Laminated	6 to 20
Very thinly bedded	20 to 60
Thinly bedded	60 to 200
Medium bedded	200 to 600
Thickly bedded	600 to 2000
Very thickly bedded	> 2000

Terms for describing rock and defects are as follows.

Defect Terms			
Joint	JT	Sheared zone	SZ
Bedding Parting	BP	Seam	SM
Foliation	FL	Vein	VN
Cleavage	CL	Drill Lift	DL
Crushed Seam	CS	Handling Break	HB
Fracture Zone	FZ	Drilling Break	DB

The shape and roughness of defects in the rock mass are described using the following terms.

Planarity		Roughness	
Planar	PR	Very Rough	VR
Curved	CU	Rough	RF
Undulose	UN	Smooth	S
Irregular	IR	Slickensided	SL
Stepped	ST	Polished	POL
Discontinuous	DIS		

The coating or infill associated with defects in the rock mass are described as follows.

Infill and Coating		
Clean	CN	
Stained	SN	
Carbonaceous	X	
Minerals	MU	Unidentified mineral
	MS	Secondary mineral
	KT	Chlorite
	CA	Calcite
	Fe	Iron Oxide
	Qz	Quartz
Veneer	VNR	Thin or patchy coating
Coating	CT	Infill up to 1mm

Graphic Symbols Index

- CLAY
- Silty CLAY
- Sandy CLAY
- Gravelly CLAY
- Silty Gravelly CLAY
- Silty Sandy CLAY
- Sandy Gravelly CLAY

- SILT
- Clayey SILT
- Sandy SILT
- Gravelly SILT
- Clayey Sandy SILT
- Clayey Gravelly SILT
- Sandy Gravelly SILT

- SAND
- Clayey SAND
- Silty SAND
- Gravelly SAND
- Clayey Silty SAND
- Clayey Gravelly SAND
- Silty Gravelly SAND

- GRAVEL
- Clayey GRAVEL
- Silty GRAVEL
- Sandy GRAVEL
- Clayey Silty GRAVEL
- Clayey Sandy GRAVEL
- Silty Sandy GRAVEL

- COBBLES & BOULDERS
- PEAT, highly organic soil
- TOPSOIL
- FILL
- FILL: Asphalt or Bituminous Seal
- FILL: Ballast
- FILL: Concrete
- FILL: Roadbase

- Sedimentary rock: fine, mostly clay (CLAYSTONE)
- Sedimentary rock: fine, mostly silt (SILTSTONE)
- Sedimentary rock: fine, silt and clay (MUDSTONE, SHALE, LAMINITE)
- Sedimentary rock: medium (SANDSTONE, GREYWACKE)
- Sedimentary rock: fine to coarse, angular (BRECCIA)
- Sedimentary rock: coarse, rounded (CONGLOMERATE)
- Sedimentary rock: Organic (COAL)
- Sedimentary rock: Carbonate (LIMESTONE, DOLOMITE)
- Sedimentary rock: Volcanic (TUFF, VOLCANIC BRECCIA, AGGLOMERATE)

- Igneous rock: Felsic, fine (RHYOLITE)
- Igneous rock: Felsic, coarse (GRANITE)
- Igneous rock: Mafic, fine to medium (BASALT, DOLERITE)
- Igneous rock: Mafic, coarse (GABBRO)
- Metamorphic rock: Foliated, fine to medium (SLATE, PHYLLITE, SHIST)
- Metamorphic rock: Foliated, coarse (GNEISS)
- Metamorphic rock: Non-foliated (QUARTZITE, HORNFELS, MARBLE)

TEST BORE LOG SHEET

Client: Griffith City Council
Project: Hanwood Stormwater Pump and Levee
Location: Hanwood, NSW

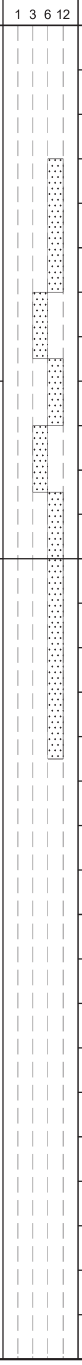
Hole No: BH001

Job No: 80518062
Sheet: 1 of 1

Position: Kidman Way shoulder. See site plan for location	**Angle from Horizontal:** 90°	**Surface Elevation:**
Machine Type: 5 tonne Excavator	**Excavation Method:** 300 mm Auger	
Excavation Dimensions:		**Contractor:** Crotty Excavations
Date Excavated: 30/4/18	**Logged By:** JB	**Checked By:** DGB

Drilling: Method	Drilling: Resistance	Drilling: Stability	Water	Sampling & Testing: Sample or Field Test	Depth (m)	Graphic Log	Classification	Material Description: SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS (0.00 – 1.20 m)		Stable	Groundwater Not Encountered	B 0.00 - 0.80 m	0.01m			ASPHALT			FILL PAVEMENT
					0.30m	▩	GC	FILL: Clayey GRAVEL; fine to coarse, sub-rounded to angular, brown-orange	D - M		
					1.20m	▨	CI-CH	Silty CLAY; medium to high plasticity, brown	M (<PL)	St - VSt	ALLUVIUM
								TERMINATED AT 1.20 m Target depth			

DCP (blows per 150 mm)



METHOD
- EX Excavator bucket
- R Ripper
- HA Hand auger
- PT Push tube
- SON Sonic drilling
- AH Air hammer
- PS Percussion sampler
- AS Short spiral auger
- AD/V Solid flight auger: V-Bit
- AD/T Solid flight auger: TC-Bit
- HFA Hollow flight auger
- WB Washbore drilling
- RR Rock roller

PENETRATION
- VE Very Easy (No Resistance)
- E Easy
- F Firm
- H Hard
- VH Very Hard (Refusal)

WATER
- ▽ Water Level on Date shown
- ▶ water inflow
- ◀ water outflow

FIELD TESTS
- SPT - Standard Penetration Test
- HP - Hand/Pocket Penetrometer
- DCP - Dynamic Cone Penetrometer
- PSP - Perth Sand Penetrometer
- MC - Moisture Content
- PBT - Plate Bearing Test
- IMP - Borehole Impression Test
- PID - Photoionisation Detector
- VS - Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES
- B - Bulk disturbed sample
- D - Disturbed sample
- ES - Environmental sample
- U - Thin wall tube 'undisturbed'

MOISTURE
- D - Dry
- M - Moist
- W - Wet
- PL - Plastic limit
- LL - Liquid limit
- *w* - Moisture content

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

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

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TEST BORE LOG SHEET

Client: Griffith City Council
Project: Hanwood Stormwater Pump and Levee
Location: Hanwood, NSW

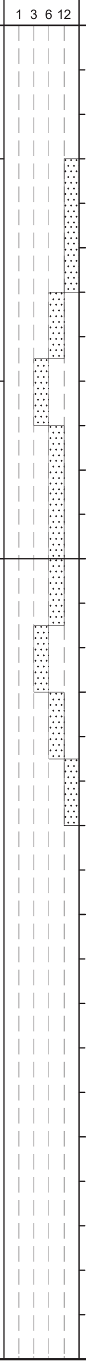
Hole No: BH002

Job No: 80518062
Sheet: 1 of 1

Position: Kidman Way shoulder. See site plan for location	**Angle from Horizontal:** 90°	**Surface Elevation:**
Machine Type: 5 tonne Excavator	**Excavation Method:** 300 mm Auger	
Excavation Dimensions:		**Contractor:** Crotty Excavations
Date Excavated: 30/4/18	**Logged By:** JB	**Checked By:** DGB

Drilling: Method	Drilling: Resistance	Drilling: Stability	Water	Sampling & Testing: Sample or Field Test	Depth (m)	Graphic Log	Classification	Material Description: SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered		0.00 - 0.01m			ASPHALT			FILL
					0.01 - 0.30m	▩	GC	FILL: Clayey GRAVEL; fine to coarse, sub-rounded to angular, brown-orange	D - M		PAVEMENT
				B 0.30 - 0.80 m	0.30 - 1.20m	▨	CI-CH	Silty CLAY; medium to high plasticity, brown 0.60 m: As above, trace root fibres 0.70 m: As above, trace roots	M (<PL)	VSt - H	ALLUVIUM
					1.20m			TERMINATED AT 1.20 m Target depth			

DCP (blows per 150 mm)



METHOD		PENETRATION		FIELD TESTS		SAMPLES		SOIL CONSISTENCY	
EX	Excavator bucket	VE	Very Easy (No Resistance)	SPT -	Standard Penetration Test	B -	Bulk disturbed sample	VS -	Very Soft
R	Ripper	E	Easy	HP -	Hand/Pocket Penetrometer	D -	Disturbed sample	S -	Soft
HA	Hand auger	F	Firm	DCP -	Dynamic Cone Penetrometer	ES -	Environmental sample	F -	Firm
PT	Push tube	H	Hard	PSP -	Perth Sand Penetrometer	U -	Thin wall tube 'undisturbed'	St -	Stiff
SON	Sonic drilling	VH	Very Hard (Refusal)	MC -	Moisture Content	**MOISTURE**		VSt -	Very Stiff
AH	Air hammer	**WATER**		PBT -	Plate Bearing Test	D -	Dry	H -	Hard
PS	Percussion sampler	▽	Water Level on Date shown	IMP -	Borehole Impression Test	M -	Moist	**RELATIVE DENSITY**	
AS	Short spiral auger	▶	water inflow	PID -	Photoionisation Detector	W -	Wet	VL -	Very Loose
AD/V	Solid flight auger: V-Bit	◀	water outflow	VS -	Vane Shear; P=Peak, R=Resdual (uncorrected kPa)	PL -	Plastic limit	L -	Loose
AD/T	Solid flight auger: TC-Bit					LL -	Liquid limit	MD -	Medium Dense
HFA	Hollow flight auger					*w* -	Moisture content	D -	Dense
WB	Washbore drilling							VD -	Very Dense
RR	Rock roller								

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

Cardno

TEST BORE LOG SHEET

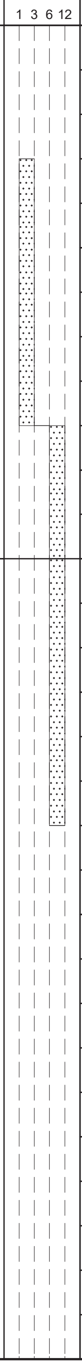
Client: Griffith City Council
Project: Hanwood Stormwater Pump and Levee
Location: Hanwood, NSW

Hole No: BH003
Job No: 80518062
Sheet: 1 of 1

Position: Kidman Way shoulder. See site plan for location	**Angle from Horizontal:** 90°	**Surface Elevation:**
Machine Type: 5 tonne Excavator	**Excavation Method:** 300 mm Auger	
Excavation Dimensions:		**Contractor:** Crotty Excavations
Date Excavated: 30/4/18	**Logged By:** JB	**Checked By:** DGB

Drilling: Method	Drilling: Resistance	Drilling: Stability	Water	Sampling & Testing: Sample or Field Test	Depth (m)	Classification	Material Description: SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered		0.02m		ASPHALT			FILL
					0.30m	GW	FILL: Sandy GRAVEL; fine to coarse, sub-rounded to angular, brown-orange	D - M		PAVEMENT
					1.20m	CI-CH	Sandy CLAY; medium to high plasticity, brown, trace gravel	M (<PL)	F - St VSt	ALLUVIUM
							TERMINATED AT 1.20 m Target depth			

DCP (blows per 150 mm)



Depth (m): 0.5, 1.0, 1.5, 2.0, 2.5

METHOD

EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION

VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

▽ Water Level on Date shown
▶ water inflow
◀ water outflow

FIELD TESTS

SPT - Standard Penetration Test
HP - Hand/Pocket Penetrometer
DCP - Dynamic Cone Penetrometer
PSP - Perth Sand Penetrometer
MC - Moisture Content
PBT - Plate Bearing Test
IMP - Borehole Impression Test
PID - Photoionisation Detector
VS - Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

B - Bulk disturbed sample
D - Disturbed sample
ES - Environmental sample
U - Thin wall tube 'undisturbed'

MOISTURE

D - Dry
M - Moist
W - Wet
PL - Plastic limit
LL - Liquid limit
w - Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

Cardno

TEST BORE LOG SHEET

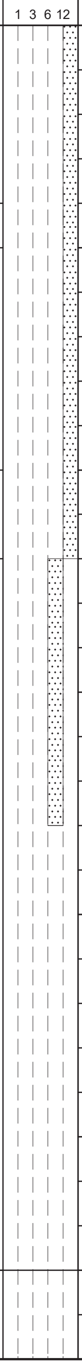
Client: Griffith City Council
Project: Hanwood Stormwater Pump and Levee
Location: Hanwood, NSW

Hole No: BH004

Job No: 80518062
Sheet: 1 of 1

Position: Proposed levee. See site plan for location	**Angle from Horizontal:** 90°	**Surface Elevation:**
Machine Type: 5 tonne Excavator	**Excavation Method:** 300 mm Auger	
Excavation Dimensions:		**Contractor:** Crotty Excavations
Date Excavated: 1/5/18	**Logged By:** JB	**Checked By:** DGB

Drilling				Sampling & Testing			Material Description					
Method	Resistance	Stability	Water	Sample or Field Test	DCP (blows per 150 mm) 1 3 6 12	Depth (m)	Graphic Log	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered				▨	GC	FILL: Clayey GRAVEL; fine to coarse grained, sub-rounded to angular, brown	D - M	VD	FILL
						0.20m	▨	CI-CH	Silty CLAY; medium to high plasticity, brown, trace gravel		VSt - H	ALLUVIUM
				D 0.40 - 0.50 m		0.5				M (<PL)		
				U50 1.00 - 1.20 m		1.0				M (≈PL)		
						1.5			1.50 m: As above, high plasticity			
						2.0				M (>PL)		
						2.5						
						2.80m			TERMINATED AT 2.80 m Target depth			



METHOD	
EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION	
VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

- ▽ Water Level on Date shown
- ▶ water inflow
- ◀ water outflow

FIELD TESTS	
SPT	Standard Penetration Test
HP	Hand/Pocket Penetrometer
DCP	Dynamic Cone Penetrometer
PSP	Perth Sand Penetrometer
MC	Moisture Content
PBT	Plate Bearing Test
IMP	Borehole Impression Test
PID	Photoionisation Detector
VS	Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES	
B	Bulk disturbed sample
D	Disturbed sample
ES	Environmental sample
U	Thin wall tube 'undisturbed'

MOISTURE	
D	Dry
M	Moist
W	Wet
PL	Plastic limit
LL	Liquid limit
w	Moisture content

SOIL CONSISTENCY	
VS	Very Soft
S	Soft
F	Firm
St	Stiff
VSt	Very Stiff
H	Hard

RELATIVE DENSITY	
VL	Very Loose
L	Loose
MD	Medium Dense
D	Dense
VD	Very Dense

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

Cardno

TEST BORE LOG SHEET

Client: Griffith City Council			**Hole No: BH005**
Project: Hanwood Stormwater Pump and Levee			
Location: Hanwood, NSW	**Job No: 80518062**		**Sheet: 1 of 1**
Position: Proposed levee. See site plan for location	**Angle from Horizontal: 90°**	**Surface Elevation:**	
Machine Type: 5 tonne Excavator	**Excavation Method: 300 mm Auger**		
Excavation Dimensions:		**Contractor: Crotty Excavations**	
Date Excavated: 1/5/18	**Logged By: JB**	**Checked By: DGB**	

Drilling				Sampling & Testing		Material Description					
Method	Resistance	Stability	Water	Sample or Field Test	Depth (m)	Graphic Log	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered		0.0 – 0.15m	▩	GC	FILL: Clayey GRAVEL; fine to coarse, sub-rounded to angular, brown	D - M		FILL
				ES 0.40 - 0.50 m	0.15m – 0.5	▨	CH	Silty CLAY; high plasticity, brown, trace sand and gravel	M (<PL)	VSt	ALLUVIUM
				B 0.80 - 1.00 m	0.5 – 1.0	▨		Fines = 84%, Sand = 7%, Gravel = 9%	M (≈PL)		
					1.0 – 1.5	▨					
					1.5 – 2.0	▨					
					2.0 – 2.5	▨					
					2.5 – 2.80m	▨					
					2.80m			TERMINATED AT 2.80 m Target depth			

METHOD

EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION

VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

- Water Level on Date shown
- water inflow
- water outflow

FIELD TESTS

- SPT - Standard Penetration Test
- HP - Hand/Pocket Penetrometer
- DCP - Dynamic Cone Penetrometer
- PSP - Perth Sand Penetrometer
- MC - Moisture Content
- PBT - Plate Bearing Test
- IMP - Borehole Impression Test
- PID - Photoionisation Detector
- VS - Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

- B - Bulk disturbed sample
- D - Disturbed sample
- ES - Environmental sample
- U - Thin wall tube 'undisturbed'

MOISTURE

- D - Dry
- M - Moist
- W - Wet
- PL - Plastic limit
- LL - Liquid limit
- *w* - Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

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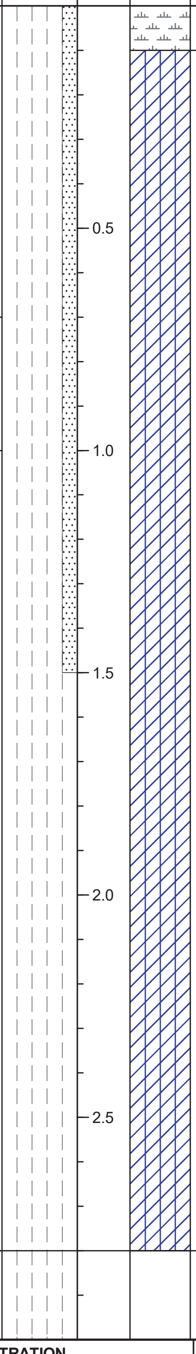
TEST BORE LOG SHEET

Client: Griffith City Council			**Hole No: BH006**
Project: Hanwood Stormwater Pump and Levee			
Location: Hanwood, NSW	**Job No: 80518062**		**Sheet: 1 of 1**
Position: Proposed levee. See site plan for location	**Angle from Horizontal: 90°**	**Surface Elevation:**	
Machine Type: 5 tonne Excavator	**Excavation Method: 300 mm Auger**		
Excavation Dimensions:		**Contractor: Crotty Excavations**	
Date Excavated: 1/5/18	**Logged By: JB**	**Checked By: DGB**	

Drilling				Sampling & Testing			Material Description					
Method	Resistance	Stability	Water	Sample or Field Test	DCP (blows per 150 mm) 1 3 6 12	Depth (m)	Graphic Log	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered					CL	TOPSOIL: Gravelly Sandy CLAY; trace root fibres	D	H	TOPSOIL
						0.10m		CI	Silty CLAY; medium plasticity, brown, trace gravel	M (<PL)	H	ALLUVIUM
				B 0.70 - 1.00 m								
									1.0 m: As above, trace root fibres	M (≈PL)		
										M (>PL)		
						2.80m			TERMINATED AT 2.80 m Target depth			



METHOD

EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION

VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

- ▽ Water Level on Date shown
- ▶ water inflow
- ◀ water outflow

FIELD TESTS

SPT	Standard Penetration Test
HP	Hand/Pocket Penetrometer
DCP	Dynamic Cone Penetrometer
PSP	Perth Sand Penetrometer
MC	Moisture Content
PBT	Plate Bearing Test
IMP	Borehole Impression Test
PID	Photoionisation Detector
VS	Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

B	Bulk disturbed sample
D	Disturbed sample
ES	Environmental sample
U	Thin wall tube 'undisturbed'

MOISTURE

D	Dry
M	Moist
W	Wet
PL	Plastic limit
LL	Liquid limit
w	Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

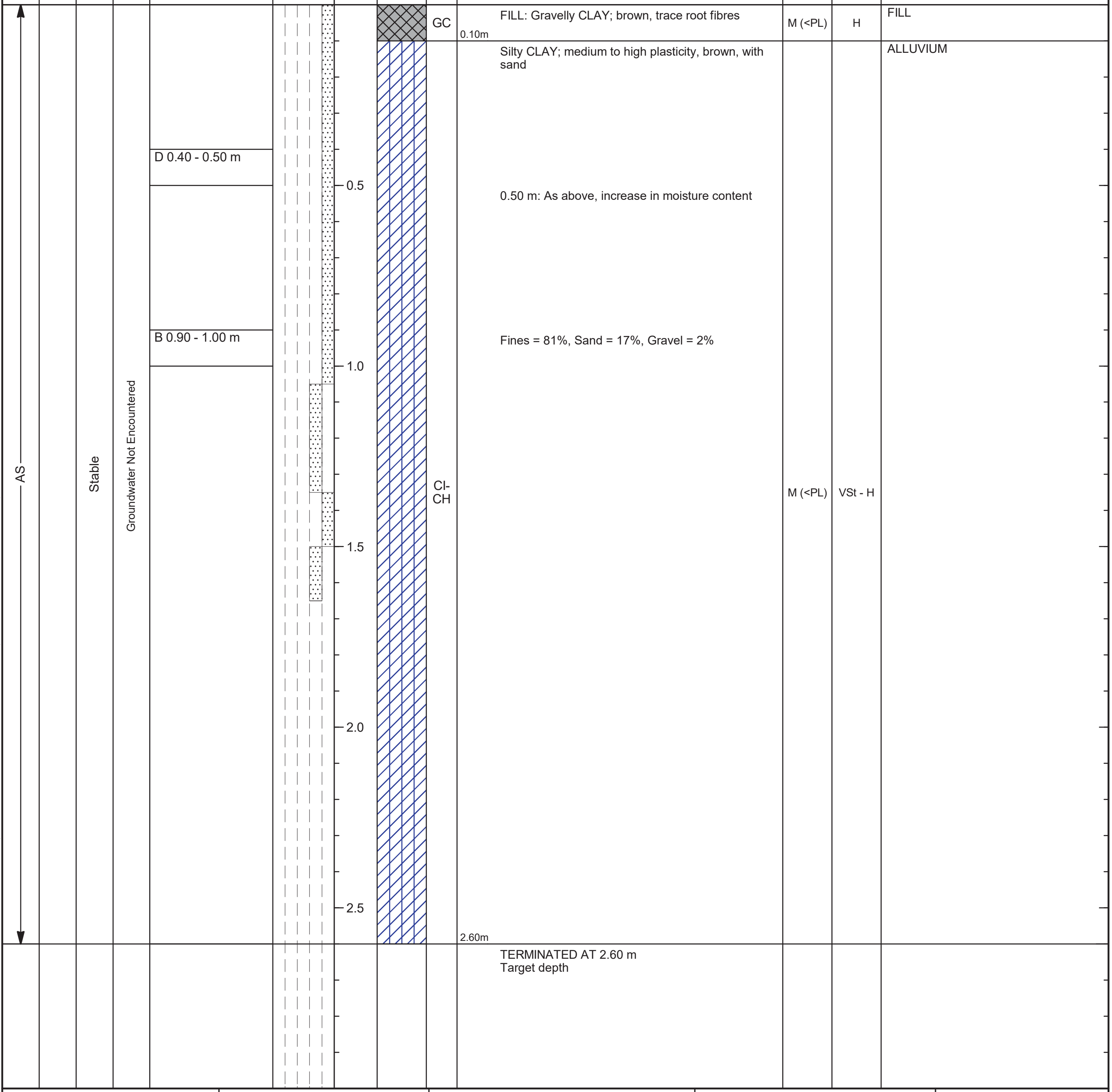
TEST BORE LOG SHEET

Client: Griffith City Council
Project: Hanwood Stormwater Pump and Levee
Location: Hanwood, NSW

Hole No: BH007

Job No: 80518062
Sheet: 1 of 1

Position: Proposed levee. See site plan for location
Angle from Horizontal: 90°
Surface Elevation:
Machine Type: 5 tonne Excavator
Excavation Method: 300 mm Auger
Excavation Dimensions:
Contractor: Crotty Excavations
Date Excavated: 1/5/18
Logged By: JB
Checked By: DGB



Method	Resistance	Stability	Water	Sample or Field Test	DCP (blows per 150 mm)	Depth (m)	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered			0.10m	GC	FILL: Gravelly CLAY; brown, trace root fibres	M (<PL)	H	FILL
				D 0.40 - 0.50 m			CI-CH	Silty CLAY; medium to high plasticity, brown, with sand	M (<PL)	VSt - H	ALLUVIUM
						0.5		0.50 m: As above, increase in moisture content			
				B 0.90 - 1.00 m				Fines = 81%, Sand = 17%, Gravel = 2%			
						2.60m		TERMINATED AT 2.60 m Target depth			

METHOD

EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION

VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

- ▽ Water Level on Date shown
- ▶ water inflow
- ◀ water outflow

FIELD TESTS

SPT	Standard Penetration Test
HP	Hand/Pocket Penetrometer
DCP	Dynamic Cone Penetrometer
PSP	Perth Sand Penetrometer
MC	Moisture Content
PBT	Plate Bearing Test
IMP	Borehole Impression Test
PID	Photoionisation Detector
VS	Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

B	Bulk disturbed sample
D	Disturbed sample
ES	Environmental sample
U	Thin wall tube 'undisturbed'

MOISTURE

D	Dry
M	Moist
W	Wet
PL	Plastic limit
LL	Liquid limit
w	Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

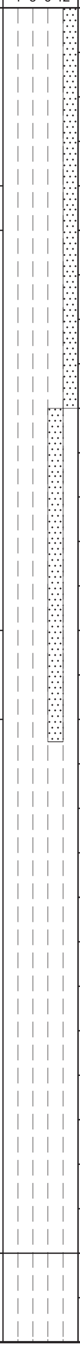
CARDNO 2.01.6 LIB.GLB Log CARDNO NON-CORED 80518062 HANWOOD LEVEE.GPJ <<DrawingFile>> 24/07/2018 08:28 10.0.000 Datgel AGS RTA, Photo, Monitoring Tools

Cardno

TEST BORE LOG SHEET

Client: Griffith City Council **Project:** Hanwood Stormwater Pump and Levee **Location:** Hanwood, NSW	**Job No: 80518062**	**Hole No: BH008** **Sheet: 1 of 1**
Position: Proposed levee. See site plan for location	**Angle from Horizontal: 90°**	**Surface Elevation:**
Machine Type: 5 tonne Excavator	**Excavation Method: 300 mm Auger**	
Excavation Dimensions:		**Contractor: Crotty Excavations**
Date Excavated: 1/5/18	**Logged By: JB**	**Checked By: DGB**

Drilling: Method	Drilling: Resistance	Drilling: Stability	Water	Sampling & Testing: Sample or Field Test	Sampling & Testing: DCP (blows per 150 mm) 1 3 6 12	Depth (m)	Material Description: Graphic Log	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered				▩	GC	FILL: Gravelly CLAY; brown, trace root fibres 0.10m	M (<PL)	H	FILL
				D 0.40 - 0.50 m		0.5	▨	CI-CH	Silty CLAY; medium to high plasticity, brown, trace gravel	M (<PL)	VSt - H	ALLUVIUM
						1.0						
				B 1.40 - 1.60 m		1.5						
						2.0				M (≈PL)		
						2.5			2.80m			
									TERMINATED AT 2.80 m Target depth			



METHOD

EX Excavator bucket
R Ripper
HA Hand auger
PT Push tube
SON Sonic drilling
AH Air hammer
PS Percussion sampler
AS Short spiral auger
AD/V Solid flight auger: V-Bit
AD/T Solid flight auger: TC-Bit
HFA Hollow flight auger
WB Washbore drilling
RR Rock roller

PENETRATION

VE Very Easy (No Resistance)
E Easy
F Firm
H Hard
VH Very Hard (Refusal)

WATER

▽ Water Level on Date shown
▶ water inflow
◀ water outflow

FIELD TESTS

SPT - Standard Penetration Test
HP - Hand/Pocket Penetrometer
DCP - Dynamic Cone Penetrometer
PSP - Perth Sand Penetrometer
MC - Moisture Content
PBT - Plate Bearing Test
IMP - Borehole Impression Test
PID - Photoionisation Detector
VS - Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

B - Bulk disturbed sample
D - Disturbed sample
ES - Environmental sample
U - Thin wall tube 'undisturbed'

MOISTURE

D - Dry
M - Moist
W - Wet
PL - Plastic limit
LL - Liquid limit
w - Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

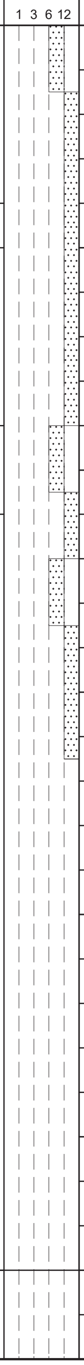
TEST BORE LOG SHEET

Client: Griffith City Council
Project: Hanwood Stormwater Pump and Levee
Location: Hanwood, NSW

Hole No: BH009

Job No: 80518062
Sheet: 1 of 1

Position: Proposed levee. See site plan for location
Angle from Horizontal: 90°
Surface Elevation:
Machine Type: 5 tonne Excavator
Excavation Method: 300 mm Auger
Excavation Dimensions:
Contractor: Crotty Excavations
Date Excavated: 1/5/18
Logged By: JB
Checked By: DGB



Drilling				Sampling & Testing			Material Description					
Method	Resistance	Stability	Water	Sample or Field Test	DCP (blows per 150 mm)	Depth (m)	Graphic Log	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered			0.10m			FILL: Silty CLAY; red-brown, trace root fibres, trace gravel	M (<PL)	VSt	FILL
				ES 0.40 - 0.50 m U50 0.90 - 1.10 m		0.5 1.0 1.5 2.0 2.5 2.80m		CI-CH	Silty CLAY; medium to high plasticity, brown, trace gravel 1.50 m: As above, increase in moisture content	M (<PL)	VSt - H	ALLUVIUM
									TERMINATED AT 2.80 m Target depth			

METHOD

EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION

VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

- ▽ Water Level on Date shown
- ▶ water inflow
- ◀ water outflow

FIELD TESTS

SPT	Standard Penetration Test
HP	Hand/Pocket Penetrometer
DCP	Dynamic Cone Penetrometer
PSP	Perth Sand Penetrometer
MC	Moisture Content
PBT	Plate Bearing Test
IMP	Borehole Impression Test
PID	Photoionisation Detector
VS	Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

B	Bulk disturbed sample
D	Disturbed sample
ES	Environmental sample
U	Thin wall tube 'undisturbed'

MOISTURE

D	Dry
M	Moist
W	Wet
PL	Plastic limit
LL	Liquid limit
w	Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

TEST BORE LOG SHEET

Client: Griffith City Council			**Hole No: BH010**
Project: Hanwood Stormwater Pump and Levee			
Location: Hanwood, NSW	**Job No: 80518062**		**Sheet: 1 of 1**
Position: Proposed levee. See site plan for location	**Angle from Horizontal: 90°**	**Surface Elevation:**	
Machine Type: 5 tonne Excavator	**Excavation Method: 300 mm Auger**		
Excavation Dimensions:		**Contractor: Crotty Excavations**	
Date Excavated: 1/5/18	**Logged By: JB**	**Checked By: DGB**	

Drilling				Sampling & Testing			Material Description					
Method	Resistance	Stability	Water	Sample or Field Test	DCP (blows per 150 mm) 1 3 6 12	Depth (m)	Graphic Log	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
AS		Stable	Groundwater Not Encountered			0.0		CI-CH	Silty CLAY; medium to high plasticity, brown, trace sand	M (<PL)	H	ALLUVIUM
				B 0.30 - 0.60 m					Fines = 85%, Sand = 12%, Gravel = 3%			
						0.5						
						1.0						
				D 1.20 - 1.40 m								
						1.5						
						2.0						
						2.5			2.60m			
									TERMINATED AT 2.60 m Target depth			

METHOD

EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION

VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

▽ Water Level on Date shown
▶ water inflow
◀ water outflow

FIELD TESTS

SPT	Standard Penetration Test
HP	Hand/Pocket Penetrometer
DCP	Dynamic Cone Penetrometer
PSP	Perth Sand Penetrometer
MC	Moisture Content
PBT	Plate Bearing Test
IMP	Borehole Impression Test
PID	Photoionisation Detector
VS	Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

B	Bulk disturbed sample
D	Disturbed sample
ES	Environmental sample
U	Thin wall tube 'undisturbed'

MOISTURE

D	Dry
M	Moist
W	Wet
PL	Plastic limit
LL	Liquid limit
w	Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD

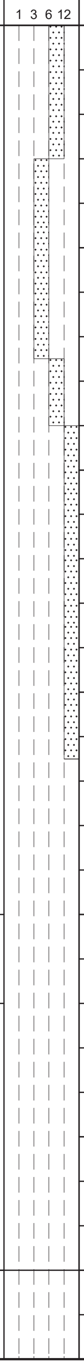
TEST BORE LOG SHEET

Client: Griffith City Council
Project: Hanwood Stormwater Pump and Levee
Location: Hanwood, NSW

Hole No: BH011

Job No: 80518062
Sheet: 1 of 1

Position: Proposed levee. See site plan for location
Angle from Horizontal: 90°
Surface Elevation:
Machine Type: 5 tonne Excavator
Excavation Method: 300 mm Auger
Excavation Dimensions:
Contractor: Crotty Excavations
Date Excavated: 1/5/18
Logged By: JB
Checked By: DGB



Depth (m)	Method	Resistance	Stability	Water	Sample or Field Test	Classification	SOIL TYPE, plasticity or particle characteristic, colour, secondary and minor components ROCK TYPE, grain size and type, colour, fabric & texture, strength, weathering, defects and structure	Moisture Condition	Consistency Relative Density	STRUCTURE & Other Observations
0.00 – 2.00m	AS		Stable	Groundwater Not Encountered		CI	FILL: CLAY; medium plasticity, pale brown	M (<PL)	St - VSt H	Possibly FILL
2.00m – 2.80m					B 2.00 - 2.20 m	CI-CH	Silty CLAY; medium to high plasticity, brown, trace gravel 2.50 m: As above, increase in moisture content	M (<PL)	VSt	Possibly ALLUVIUM
2.80m							TERMINATED AT 2.80 m Target depth			

METHOD

EX	Excavator bucket
R	Ripper
HA	Hand auger
PT	Push tube
SON	Sonic drilling
AH	Air hammer
PS	Percussion sampler
AS	Short spiral auger
AD/V	Solid flight auger: V-Bit
AD/T	Solid flight auger: TC-Bit
HFA	Hollow flight auger
WB	Washbore drilling
RR	Rock roller

PENETRATION

VE	Very Easy (No Resistance)
E	Easy
F	Firm
H	Hard
VH	Very Hard (Refusal)

WATER

- ▽ Water Level on Date shown
- ► water inflow
- ◄ water outflow

FIELD TESTS

SPT -	Standard Penetration Test
HP -	Hand/Pocket Penetrometer
DCP -	Dynamic Cone Penetrometer
PSP -	Perth Sand Penetrometer
MC -	Moisture Content
PBT -	Plate Bearing Test
IMP -	Borehole Impression Test
PID -	Photoionisation Detector
VS -	Vane Shear; P=Peak, R=Resdual (uncorrected kPa)

SAMPLES

B -	Bulk disturbed sample
D -	Disturbed sample
ES -	Environmental sample
U -	Thin wall tube 'undisturbed'

MOISTURE

D -	Dry
M -	Moist
W -	Wet
PL -	Plastic limit
LL -	Liquid limit
w -	Moisture content

SOIL CONSISTENCY

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

RELATIVE DENSITY

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

Refer to explanatory notes for details of abbreviations and basis of descriptions

CARDNO (NSW/ACT) PTY LTD