North Tuncurry Urban Release Area

Addendum report to the Integrated Water Cycle Management Strategy

Prepared for Landcom
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Addendum report to the Integrated Water Cycle Management Strategy

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1 Introduction

1.1 Background

The North Tuncurry Urban Release Area (NTURA or the project) is a proposed residential development sponsored by Landcom under a Project Delivery Agreement with the Crown Lands Branch of NSW Department of Planning, Industry and Environment, who control the land. The project is located to the north of Tuncurry on 615 ha of land.

The project masterplan and technical studies were completed over the 2012-2014 period in consultation with (the then) Great Lakes Council and other government agencies. A rezoning proposal was initially submitted to the (then) Department of Planning in 2014 for review prior to public exhibition. The rezoning proposal was updated to address some biodiversity related concerns and was re-lodged in 2019. The rezoning proposal included the following reports that relate to stormwater and flooding:

- Integrated Water Cycle Management Strategy – Revision 5 (SMEC 2019) – Appendix P of the rezoning proposal. This report is referred to as the IWCMS in the remainder of this document.
- Groundwater modelling technical report – Revision B (SMEC 2014) – Appendix J of the rezoning proposal. This report is referred to as the GWMR in the remainder of this document.

In 2020, EMM Consulting Pty Limited (EMM) were engaged to assist Landcom address concerns raised by MidCoast Council (Council), the Department of Planning, Industry and Environment – Water (DPIE-Water) and Department of Planning, Industry and Environment – Biodiversity and Conservation Division (BCD) regarding the stormwater and flooding aspects of the proposal. While some issues raised were addressed via responses and further descriptions, additional information is required to address residual concerns. This addendum report has been prepared to provide this additional information and forms part of an updated rezoning proposal.

1.2 Addendum report scope and structure

This report is an addendum to the IWCMS. Table 1.1 provides a summary of the report scope, reason for providing further information and a chapter reference for each scope item. A summary of departures from the IWCMS is provided in Section 1.3.

Table 1.1 Addendum report scope

Aspect	Scope and reason for further information	Chapter references
Gravity drainage system	A gravity drainage system (the gravity drain) is proposed to manage surplus water from the water management basins, reducing the build-up of large volumes of water during prolonged periods of wet weather and providing a reduction in peak flood levels and durations during shorter more intense rainfall events. The gravity drain is conceptually described in the IWCMS as linking the southernmost basin to the Wallis Lake Entrance Channel. The concept has been further developed to address concerns raised by Council and BCD. Key changes from the concept presented in the IWCMS include: • the outlet concept has been revised to a surcharge pit arrangement, to prevent tidal water ingress into the system; • further information on the civil design concept and maintenance regimes is provided; and • further information on the pipe hydraulics is provided.	Chapter 2

Table 1.1 Addendum report scope

Aspect	Scope and reason for further information	Chapter references
Design storm analysis	Design storm analysis has been undertaken to complement water balance modelling[1] that was used in the IWCMS to assess flood risk. The design storm analysis assesses a range of Annual Exceedance Probability (AEP) storm events that have durations ranging from a few hours to 7 days. Analysis of the Probable Maximum Flood (PMF) is also provided. The results are used to complement the water balance results and assess flood conditions associated with > 1% AEP events and the sensitivity to key model parameters.	Chapter 3
Flood risk management strategy	The flood risk management strategy has been updated to consider the results from the water balance model, detailed groundwater model and design storm analysis.	Chapter 4
Concept designs	Describes a framework to progress the design of key elements of the water management system.	Chapter 5
Water regulation	Water licencing regulations and the NSW Aquifer Interference Policy are re-addressed in this addendum report. This has been done due to feedback from DPIE-Water who identified some errors in the information provided in IWCMS Section 2.3 and Appendices A and D.	Chapter 6

1.3 Summary of departures

Some information presented in this addendum report supersedes information presented in the IWCMS and GWMR. Table 1.2 provides a summary of departures and notes report section references to the revised information in this addendum report and the superseded information in the IWCMS and/or the GWMR. It is noted that there are no changes to the site-specific data, models or model results presented in IWCMS and the GWMR. There is also no change to the project masterplan.

Table 1.2 Summary of departures

Aspect	Description of departure	Addendum report reference	IWCMS/GWMR reference
Description of the gravity drainage system	A revised description of the gravity drain concept is provided in this addendum report (see Table 1.1 for further information). There is no change to the drain's capacity or functionality hence the drain's representation in modelling presented in the IWCMS and GWMR does not require revision.	Chapter 2	IWCMS Section 5.3.4
Flood risk management strategy	The flood risk management strategy has been updated to be more comprehensive. There is no material change to the strategy elements described in the IWCMS.	Chapter 4	IWCMS Section 5.5 and 5.6 and Table 10-1
Water licencing regulations and the NSW Aquifer Interference Policy	Water licencing regulations and the NSW Aquifer Interference Policy are re-addressed in this addendum report, superseding information provided in IWCMS.	Chapter 6 and Appendix D	IWCMS Section 2.3 and Appendices A and D

[1] The water balance model is referred to as the Empirical Groundwater Model in the IWCMS and GWM reports

2 Revised gravity drain concept

2.1 Overview

A gravity drainage system (the gravity drain) is proposed to manage surplus water from the proposed water management basins, reducing the build-up of large volumes of water during prolonged periods of wet weather and providing a reduction in peak flood levels and durations during shorter more intense rainfall events.

The gravity drain is conceptually described in the IWCMS as linking the southernmost basin to the Wallis Lake Entrance Channel. This would require the construction of a drainage system that is approximately 2 km long. The pipe is described in the IWCMS as having an inlet level of 3 m AHD, an average grade of 0.2 to 0.3% and an outlet level between -1 to -3 m AHD, depending on grade. A 1,050 mm diameter pipe was calculated to provide sufficient capacity to limit the peak 1% AEP level (calculated using the water balance model) in the southern basins to below 4 m AHD. Water balance model results presented in IWCMS Plate 5-10 show that for the simulated 1963 event (which was established as being greater than a 1% AEP event - see GWMR Section 7) the pipe would drain between 5 to 86 ML/day of water (the rate increases with higher water levels in the basin), totalling 974 ML over a period of approximately 69 days.

The gravity pipe concept has been further developed to address concerns raised by Council and BCD. Key changes from the concept presented in the IWCMS include:

- the outlet concept has been revised to a surcharge pit arrangement, to prevent tidal water ingress into the system;
- further information on the civil design concept and maintenance requirements are provided; and
- further information on the pipe hydraulics is provided.

This chapter describes the revised concept and hydraulic analysis and is structured as follows:

- Section 2.2 describes the revised concept;
- Section 2.3 describes the hydraulic analysis that has been undertaken to establish the pipe capacity and demonstrate design flexibility;
- Section 2.4 provides a summary of identified constraints, associated issues and the proposed approach; and
- Section 2.5 describes alternatives that were considered in the master planning process.

2.2 Revised concept

The revised concept is for a new drainage system to be constructed from the southernmost basin to the Wallis Lakes Entrance Channel. The preferred alignment is along Beach Street, with two possible alignments for the downstream (southern) portion of the pipe (see Section 2.2.2). The system will be standalone, meaning that there will be no stormwater inflows along the alignment, and will outlet into the Wallis Lake Entrance Channel via a surcharge pit arrangement. The pipe will be constructed as shallow as possible to minimise excavation depths and the depth of the pipe at the outlet. This will result in an average pipe grade of around 0.15%.

Figure 2.1 provides an overview of the proposed concept. The following sections describe key features in further detail.

It is noted that this information is presented to demonstrate proof of concept. The concept will be further developed at detailed design.

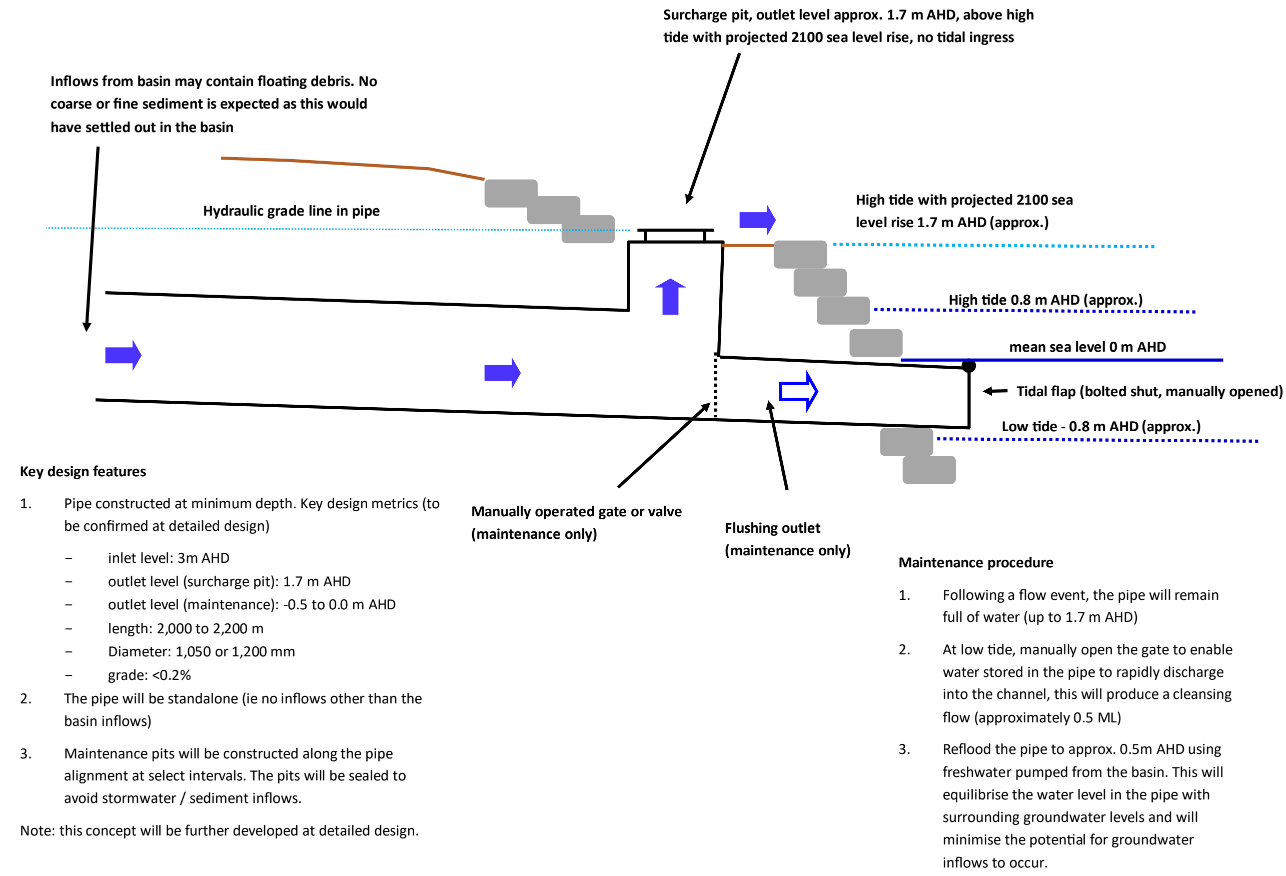

Figure 2.1 Gravity drain concept

2.2.1 Expected flow regimes

As described in IWCMS Chapter 5, flow through the pipe will only occur occasionally following periods of prolonged rainfall or significant storm events. Water balance model results presented in IWCMS Plate 5-8 indicate that flows will occurs in 40% of years. However, annual flows of greater than 100 ML/year will only occur in 12% of years.

Once flow commences, it is expected to occur contiguously (mostly at low flow rates) until wet conditions ease. Typical flow durations are expected to range from several weeks to several months. Plate 5-10 in the IWCMS shows the simulated flow regime during the 1963 event. No flow through the pipe is expected during periods of below average and average rainfall, which can occur for several years. During this time, water in the pipe will be stagnant.

2.2.2 Pipe alignment

Landcom engaged Lidbury Summers Whiteman (LSW) to review local constraints associated with existing services and land ownership and identify several possible alignments. LSW identified two alignments that have manageable constraints and are amenable to the gravity drain concept that is described in this chapter. The alignments are:

- Option 1 – the upstream (ie northern) portion of the pipe will be located in the eastern verge of Beach Street. The downstream (ie southern) portion of the pipe will be aligned through the eastern most road in the Tuncurry Holiday Park, which is operated by Reflections Holiday Parks but is located on crown land.

- Option 2 – The majority of the pipe length will be located within the eastern verge of Beach Street. The downstream or southernmost portion of the pipe will be aligned adjacent to an existing easement (on crown land).

Preliminary design drawings for each option are provided in Appendix A. The drawings include information on identified service constraints and landownership details. A preferred alignment will be selected (in consultation with stakeholders) and progressed at detailed design.

It is noted that both alignments are fully located in either the NUTRA site, a public road reserve or crown land. No unavoidable requirements for works on freehold land were identified by the preliminary design.

2.2.3 Inlet concept

The pipe inlet is conceptualised to have an invert level of 3 m AHD, although levels between 2 and 3 m AHD could be considered at detailed design. Inflows into the pipe will only occur intermittently when the water level in the basins exceeds the invert level. When this occurs, water in the basins will move slowly towards the pipe inlet. Any coarse or fine suspended material that enters to basins via stormwater inflows is expected to settle from the water column near the stormwater outlet locations. Hence, water in the basins and inflows into the gravity pipe are likely to have negligible concentrations of coarse or fine suspended sediment (ie suspended material that could potentially settle out in the pipe). Floating debris including small organic matter, reeds, small to large woody debris and floating litter are likely to be drawn towards the inlet. As a result, the inlet design will include:

- appropriate controls to prevent large floating debris from entering the pipe system and smaller floating debris from clogging any inlet screens; and

- provisions to enable maintenance access to the inlet during a full range of basin level conditions.

It is expected that these objectives can be achieved at detailed design using:

- an initial control (such as floating debris boom or a permeable rock weir) located upstream of the inlet to capture the majority of floating debris; and

- appropriate screening around the pipe inlet to prevent medium to large debris entering the system.

2.2.4 Outlet concept

The pipe outlet configuration is shown in Figure 2.1. It comprises a primary outlet via a surcharge pit arrangement and a flushing outlet for maintenance purposes. These outlets are described further below.

i Surcharge pit outlet

A surcharge pit arrangement is proposed to avoid tidal water ingress into the drainage system. The pit will be located near the Wallis Lake Entrance Channel rock revetment wall and will discharge via surcharging or upwelling. The pit overflow level will be finalised at detailed design but for the purposes of this assessment it is assumed to be at 1.7 m AHD, which is conservatively above the High High Water Solstice Spring tide, incorporating projected 2100 sea level rise (this is discussed further in Section 2.3.1).

The pit and grate size will be established at detailed design.

ii Maintenance outlet

As a result of the surcharge pit discharge arrangement, the pipe will not be free draining and the lower half (up to 1.7 m AHD) of the pipe will remain full of water following a flow event. A maintenance outlet is proposed to enable the pipe system to be manually flushed with minimal effort. The outlet will be constructed at a similar level to the base of the surcharge pit (ideally near or above low tide) and will include a manually operated valve. The pipe system can be flushed by opening the valve at low tide, allowing water stored in the pipe (approximately 0.5 ML) to rapidly flow out. This will produce a self-cleansing flow that will remove any material that has accumulated in the bottom of the pipe and surcharge pit. The maintenance outlet will be sized at detailed design.

Following flushing, the pipe can be refilled to around 0.5 m AHD with fresh water from the NTURA basins (via pumping). The refilled pipe would have a water level that is greater than the adjoining groundwater level, which is likely to be slightly higher than mean sea level. Following filling, the water level in the pipe would be expected to slowly equilibrise with the surrounding groundwater system, which will prevent any material groundwater inflows occurring into the gravity pipe. As an alternative, the pipe could be partly flushed (ie using water stored in the pipe between 1.7 and 0.5 m AHD. If this is effective, it would avoid the need to refill the pipe.

A flushing/maintenance procedure is provided in Figure 2.1.

If the projected 0.91m of sea level rise occurs, the maintenance outlet would be permanently submerged, even at low tide. Under these conditions, the flushing arrangement is still expected to be effective as the water level in gravity pipe will initially be at the surcharge pit level (1.7 m AHD), which will be approximately 1.6 m above a low tide level applying projected 2100 sea level rise (0.1 m AHD). This positive head will enable a sustained flushing flow to occur, even if the flushing outlet is submerged. The flushing outlet will need to be shut as the water level in the pipe approaches the water level in the entrance channel. If required, the remaining water in the pipe could be removed by pumping, but there is no clear and obvious reason to suspect that this will be required.

2.2.5 Pipe material options

Stormwater drainage systems are typically constructed using precast reinforced concrete pipes. Polyethylene pipes may provide a superior solution for this application as they:

- have lower roughness which can reduce pipe size and increase the flow velocity (and self-cleansing potential);
- are resistant to corrosion; and
- may provide more design and construction flexibility.

Polyethylene pipes will be considered at detailed design.

2.3 Hydraulic analysis

Hydraulic analysis of the proposed gravity drain was undertaken using HY-8 culvert hydraulic analysis software. The objective of the analysis was to assess the culvert capacity for a range of tailwater conditions, pipe roughness assumptions and pipe sizes. The results from these scenarios are used to demonstrate the capacity of the gravity drain concept.

Section 2.3.1 describes the water levels in the Wallis Lake Entrance Channel that have been applied to the analysis and Section 2.3.2 describes the assumptions and results.

2.3.1 Water levels in the Wallis Lake Entrance Channel

Information on the water levels in the Wallis Lake Entrance Channel near the gravity drain outlet are provided in Table 2.1 for a range of tide and flood conditions. Levels for both current and potential 2100 sea level conditions are provided.

The tidal statistics for current conditions were sourced from *Coastal Processes Report: Hydrodynamic and sediment transport: Assessment of Wallis Lake Dredging* (WorleyParsons 2013). This report provides information on the tidal planes and ranges at water level gauges located in the ocean, the entrance channel and Wallis Lake. Statistics from the entrance gauge, located approximately 500 m to the east of the gravity drain outlet, are considered to be the most representative of conditions at the outlet and are therefore provided in Table 2.1. However, these statistics moderately overstate the peak tide levels as the tidal range (ie difference between low and high tide levels) declines rapidly in the channel. For example, the mean spring tide range is 1.32 m in the ocean, 1.07 m at the entrance gauge (located 500 m east of the pipe outlet) and 0.24 m at the islands gauge (located at the western end or lake side of the entrance channel, approximately 5 km to the west of the pipe outlet) (WorleyParsons 2013).

The flood level information was sourced from flood maps in the *Wallis Lake Foreshore (floodplain) risk Management Study and Flood Study Review* (WMA 2014).

Table 2.1 Water level statistics – Wallis Lake Entrance Channel

	Current conditions (m AHD)	2100 sea level rise scenario[1] (m AHD)
Tidal statistics (from MHL gauge at entrance, 500 m to the east of the gravity drain outlet)		
Tidal range	1.07 m	1.07 m
High High Water Solstice Spring (HHWS)	0.80[2]	1.71
Mean High Water Spring (MHWS)	0.49[2]	1.40
Mean High Water (MHW)	0.39[2]	1.30
Mean sea level (MSL)	-0.05[2]	0.86
Mean Low Water (MLW)	-0.48[2]	0.43
Mean Low Water Springs (MLWS)	-0.58[2]	0.33
Indian Spring Low Water (LSLW)	-0.80[2]	0.11
Flood conditions		
5% AEP flood level	1.4[3]	2.3
1% AEP flood level	1.8[3]	2.7
PMF level	4.2[3]	Expected to be similar to 4.2

Notes: 1. Calculated by applying 0.91 m to the current condition value
2. Sourced from *Coastal Processes Report: Hydrodynamic and sediment transport: Assessment of Wallis Lake Dredging* (WorleyParsons 2013)
3. Sourced from flood maps provided in the *Wallis Lake Foreshore (floodplain) risk Management Study and Flood Study Review* (WMA 2014)

2.3.2 Analysis

This section describes the hydraulic modelling assumptions, scenarios and results.

i Assumptions

Key model assumptions are provided in Table 2.2. A range of values were assessed for some parameters. The values labelled as default were applied to adopted model results while values labelled as 'sensitivity analysis' were only used to test the sensitivity of a range of parameter values.

Table 2.2 Gravity pipe – hydraulic model assumptions

	Applied value or range	Comments
Inlet level	**All scenarios:** 3 m AHD	Levels between 2 and 3 m AHD could be considered at detailed design. A lower level will increase capacity, especially at lower basin levels.
Outlet level	**All scenarios:** nominal level of -0.5 m AHD	Note, the surcharge pit levels are represented as a tailwater condition.
Pipe length	**All scenarios:** 2,000 m	A longer pipe would reduce capacity, while a shorter pipe would increase capacity
Pipe diameter	**Default:** 1.05 m **Sensitivity analysis:** 1.20 and 1.35 m	Note, 1.20 and 1.35 m diameter pipes are included for sensitivity analysis only.
Pipe type	**All scenarios:** circular reinforced concrete pipe (RCP)	Note, polypropylene pipe solutions could also be considered at detailed design.
Pipe roughness	Manning's roughness values **Default:** 0.012 **Sensitivity analysis:** 0.015	0.012 is a typical Manning's roughness value used for concrete pipe. 0.015 is a value used for a rough concrete pipe and is a conservative value.
Pit losses	**All scenarios:** None applied	Pit losses are likely to be minor compared to pipe friction losses. The combined pipe and pit losses can be offset by a larger pipe diameter if needed (this is discussed below this table).
Head water condition	**All scenarios:** 3 to 5 m AHD	Note, the proposed 1% AEP level in the basins is 4.2 m AHD
Tailwater condition	**Default**: 2.0 m AHD **Sensitivity** 0.0 to 3.0 m AHD	Notes: • The default level of 2.0 m AHD accounts for the surcharge pit overflow level of 1.7 m AHD and 0.3 m of head to enable overflow from the pit to occur as weir flow. • Tailwater conditions below 2 m AHD are provided for context only.

ii Tailwater scenarios

The default pipe scenario (ie a 1,050 mm pipe with a 0.012 manning's roughness value) was applied to assess tailwater levels between 0 to 3 m AHD. The default tailwater condition is 2.0 m AHD. This accounts for the surcharge pit overflow level of 1.7 m AHD and 0.3 m of head to enable overflow from the pit to occur as weir flow. It is noted that tailwater levels below 1.5 m AHD are provided for context only.

Results are presented as rating curves in Figure 2.2. Rating curves describe the pipe's capacity at different basin (or head water) levels. The pipe inlet level, 1% AEP level and minimum floor levels are shown for context. Refer to Chapter 4 for further information on design flood levels.

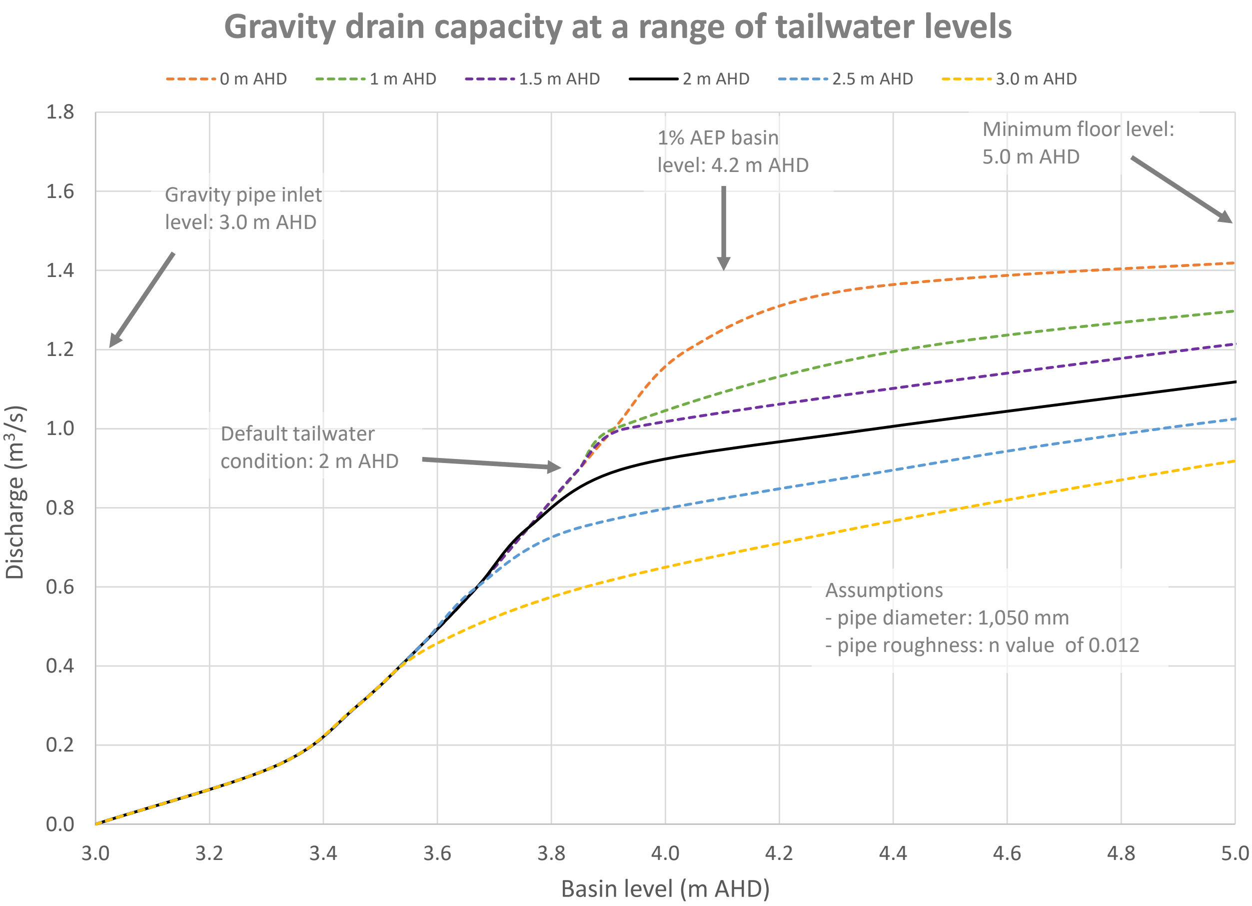

Figure 2.2 Gravity drain capacity – tailwater sensitivity

The results shown in Figure 2.2 indicate that the pipe capacity is not sensitive to the tailwater condition for flows up to 0.4 m^3/s, which occur when the basin level is within the 3 to 3.5 m AHD range. Under these flow conditions, the pipe capacity is governed solely by entrance and/or pipe friction losses in the upper portion of the pipe.

When the flow rate exceeds 0.4 m^3/s, the highest tailwater condition (3 m AHD) begins to reduce the pipe's capacity (ie the pipe capacity is governed by both friction losses and the tailwater condition). The default tailwater condition of 2 m AHD does not diverge from the no tailwater condition (0.0 m AHD) curve until flows exceed 0.8 m^3/s. At the 1% AEP basin level, the peak flow for the default tailwater condition is greater than 0.9 m^3/s, and below the 1.3 m^3/s for the no tailwater condition. This analysis demonstrates that the surcharge pit arrangement will not materially constrain (relative to the no tailwater condition) the pipe capacity below a basin level of 3.9 m AHD, but will moderately reduce the capacity above this level.

The 1% AEP level in the Wallis Lake Entrance Channel at the pipe outlet location is estimated to be 2.7 m AHD for the 2100 sea level rise scenario (see Table 2.1). Should these conditions coincide with peak flooding in the NTURA basins, the pipe capacity would be temporarily reduced from 0.9 m^3/s to 0.7 m^3/s. This is not expected to materially impact the flood levels in the NTURA basins as peak levels in the entrance channel will only occur for several hours when peak flooding conditions coincide with a high tide. For example, a 0.2 m^3/s reduction in outflow via the gravity drain over a 4-hour period equates to less than 3 ML of water, which is less than 1% of the flood storage volume provided in the basins (see Section 3.2.4).

iii Pipe size and roughness scenarios

The gravity drain concept will be further developed at detailed design, and as noted in Table 2.2, the hydraulic analysis does not make allowance for pit losses or higher than expected pipe roughness. Sensitivity analysis has been undertaken to demonstrate that the required system capacity could be achieved with a larger diameter pipe if system losses (ie pipe friction and pit losses) are higher than allowed for in the analysis presented in this report. The sensitivity analysis considers 1,050, 1,200 and 1,350 mm diameter pipes with assumed manning's roughness values of 0.012 (default) and 0.015 (conservative). The rating curves for each scenario are provided in Figure 2.3.

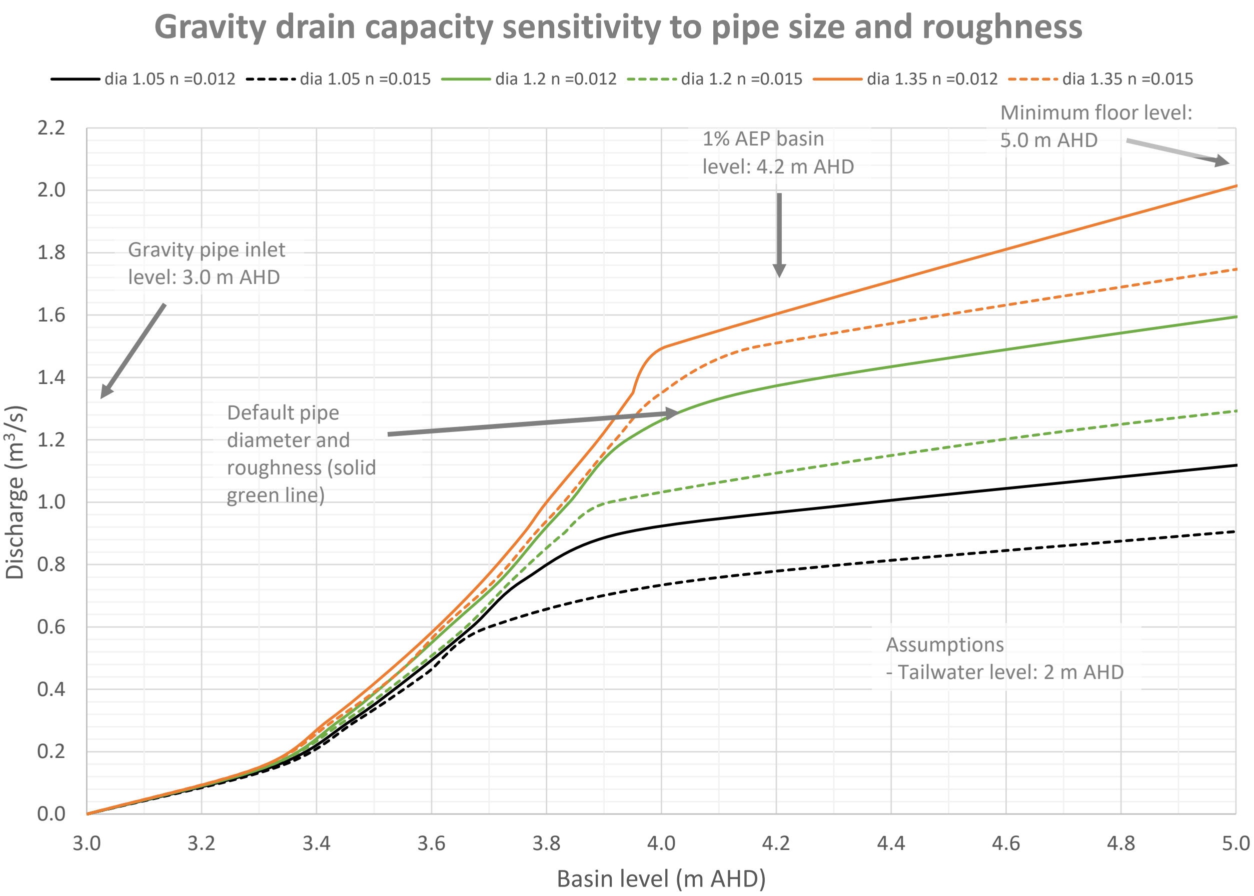

Figure 2.3 Gravity drain capacity – pipe size and roughness sensitivity

The sensitivity analysis results presented in Figure 2.3 shows that capacity of a 1,200 mm RCP with conservative roughness values is greater than the capacity of a 1,050 mm RCP with the default roughness values. This demonstrates that the required system capacity can be achieved with a larger diameter pipe if system losses (ie pipe friction and pit losses) are higher than allowed for in the analysis presented in this report.

iv Self-cleansing flows

The ability of a pipe to self-cleanse is a function of the flow velocity along the base of the pipe. A flow velocity of 0.7 m/s is commonly used as a self-flushing flow velocity in the design of gravity sewer systems.

Hydraulic analysis indicates that for flow conditions up to 0.9 m^3/s (which will only occur when the basin is at the 1% AEP level) flow through the initial section of pipe will be part full, with pipe full flow occurring in the lower section of the pipe. This is illustrated in Figure 2.4 which shows a hydraulic profile for a low flow (0.2 m^3/s) condition. Calculated velocities for part full and pipe full flow conditions are provided.

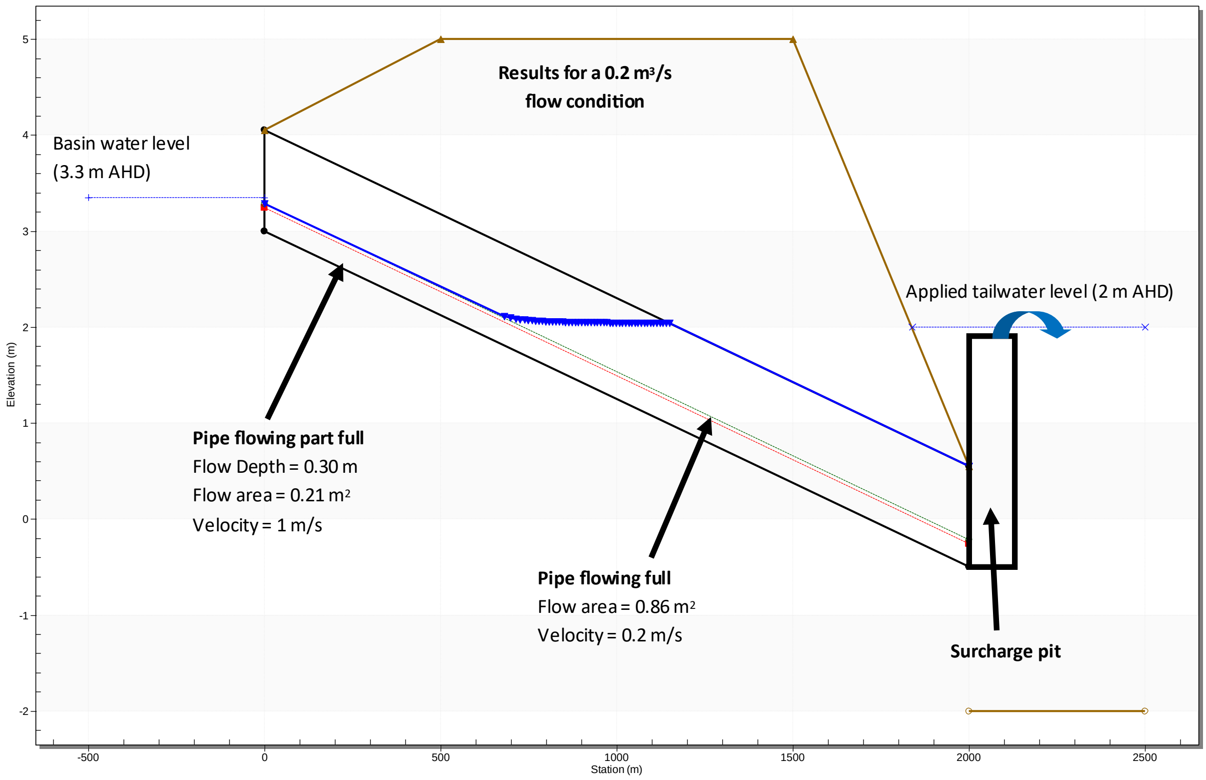

Figure 2.4 Hydraulic profile: low flow condition

The results in Figure 2.4 indicate that for low flow conditions, part full flows in the initial pipe section would have a self-cleansing velocity. The velocity will reduce in the lower pipe section where pipe full flow will occur due to the backwater effect created by the surcharge pit. The flushing arrangement described in Section 2.2.4 will effectively remove any debris that accumulate in the lower portion of the pipe and if required, a modest flow of 0.2 m^3/s could be pumped into the pipe (when the maintenance outlet is open) to achieve a full pipe flush.

v Summary

The capacity of the gravity pipe is represented in both the water balance model (described in the IWCMS) and design storm analysis (see Chapter 3) using a rating curve. The hydraulic analysis presented in this section has calculated a rating curve for the proposed gravity pipe concept and demonstrated that the stated capacity can be achieved by increasing the pipe size if system losses are higher than allowed for in the analysis presented in this report.

Figure 2.5 shows the calculated rating curve that has been applied to the design storm analysis and the rating curve that was calculated (in 2012) for use in the water balance model (see GWMR Plate 4-8).

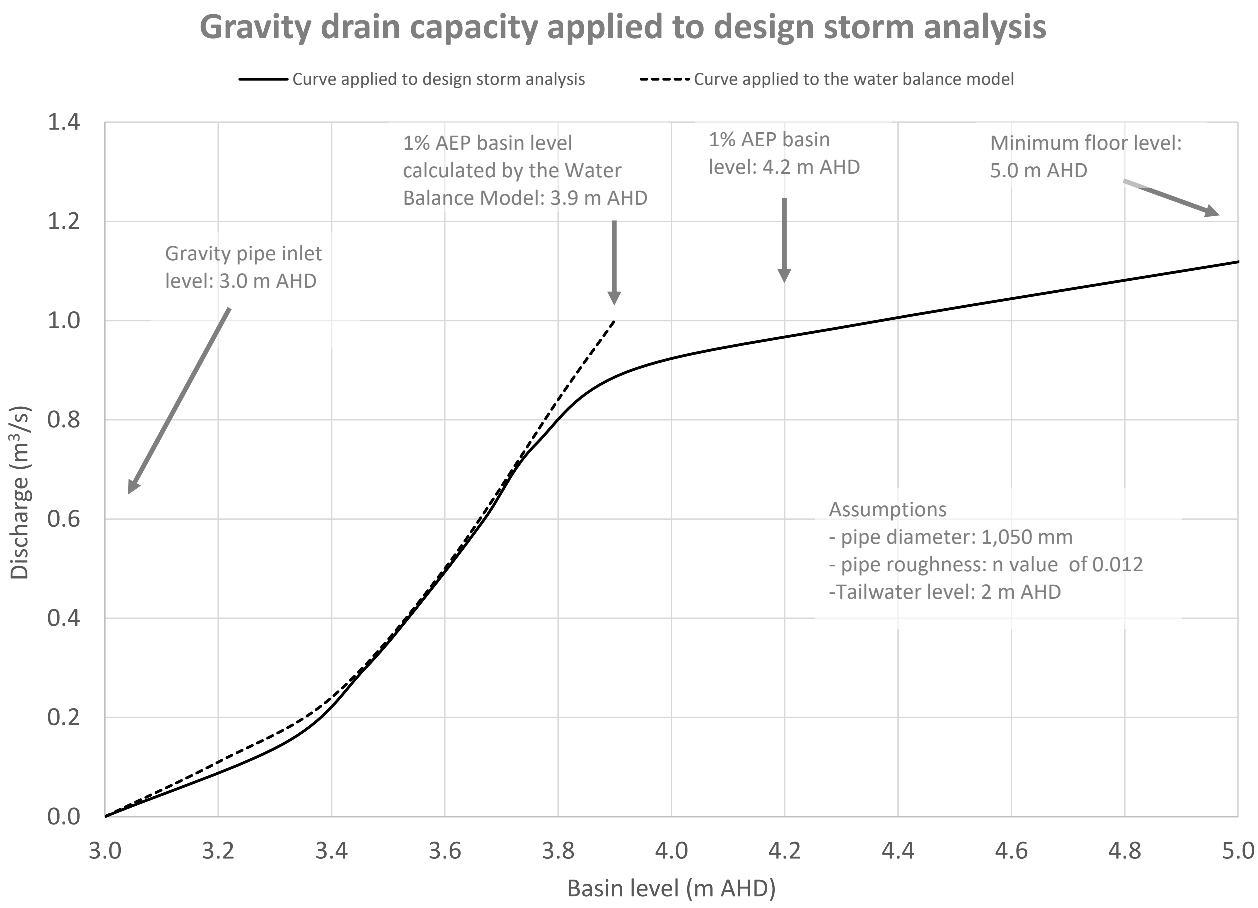

Figure 2.5 Gravity pipe rating curve applied to design storm analysis

The two curves shown in Figure 2.5 are near identical for basin water levels between 3.0 and 3.8 m AHD, but diverge for basin water levels between 3.8 and 3.9 m AHD. This divergence is due to a higher tailwater condition being applied in the current analysis. This divergence is not considered an issue as the highest simulated basin level in the water balance model is 3.9 m AHD (see IWCMS Plate 5-10) and there is only a 0.1 m^3/s difference in the calculated flow rate at this basin level. Accordingly, the gravity pipe capacity is reliably represented in the water balance model.

2.4 Constraints analysis

The gravity drain design concept has numerous constraints associated with its limited grade, outlet into an estuarine water body and its alignment through an existing urban area. The concept presented in this chapter seeks to address each constraint. Table 2.3 provides a summary of identified constraints, associated issues and the proposed approach.

Table 2.3 Gravity pipe – constraints and design approach

Constraint	Potential issues	Design approach
Constraint 1 – Discharge into estuarine waterbody	**Issue 1.1 –** Marine fouling and sedimentation issues associated with tidal inflows into the piped drainage system may reduce the pipe capacity and require onerous maintenance.	**Approach 1.1 –** The pipe system will discharge to the Wallis Lake Entrance Channel via a surcharge pit located adjacent to the channel (see Figure 2.1). The pit overflow level will be finalised at detailed design but for the purposes of this assessment it is assumed to be at 1.7 m AHD. This level is conservatively above the High High Water Solstice Spring tide, incorporating projected 2100 sea level rise (see Section 2.3.1). Hence, no tidal inflows into the drainage system are expected for current and projected 2100 sea level conditions and therefore no marine fouling or sedimentation issues are expected.
	Issue 1.2 – A deep pipe outlet into an estuarine water body would be difficult to construct and maintain.	**Approach 1.2 –** The pipe will be constructed as shallow as possible to minimise construction and maintenance complexity. The invert of the pipe at the surcharge pit is expected to be near the low tide level.
Constraint 2 – Sea level rise	**Issue 2.1** – Sea level rise could potentially result in tidal inflows into the piped drainage system and constrain the hydraulic capacity of the system.	See approach 1.1.
Constraint 3 – Pipe grade is below the recommended minimum grade of 0.3 to 0.5% for stormwater systems.	**Issue 3.1 –** Potential sediment and debris accumulation within the pipe could reduce pipe capacity and require onerous maintenance.	**Approach 3.1 –** Given the size of the NTURA basins, water that flows into the pipe from the basins is not expected to have any suspended coarse or fine sediment that may accumulate within the pipe system. However, some floating debris and organic matter are expected to enter the pipe system (see Section 2.2.3). Sediment and debris accumulation risks will be managed by: • the inlet design (offtake level and screening); • avoiding potentially sediment laden stormwater inflows along the pipe alignment; • avoiding box culverts (due to the flat bottom); • self-cleansing flows that will occur in the upper pipe section during low and greater flow conditions (see Section 2.3.2); and • provision for manually operated pipe flushing after flow events.
	Issue 3.2 – There is potential for localised sections of flat or even negatively graded pipe to occur due to construction tolerances and differential settlement post construction.	**Approach 3.2 –** Localised sections of flat or negatively graded pipe will not impact the hydraulic capacity of the pipe as the capacity is a function of the difference in water level between the inlet and outlet and system losses.

Table 2.3 Gravity pipe – constraints and design approach

Constraint	Potential issues	Design approach
Constraint 4 – Intermittent flow regime	**Issue 4.1** – Flows through the pipe will only occur occasionally, resulting in stagnant water sitting in the pipe for potentially several years. Groundwater inflows into the pipe may occur through pipe joints.	**Approach 4.1** – A maintenance procedure has been established to flush the pipe after a flow event and re-fill the pipe with clean water from the basins. It is noted that this maintenance may not be required after each event if there is minimal debris accumulation in the pipe.
Constraint 5 – Landownership and existing services	**Issue 5.1 –** The proposed pipe will need to be constructed on land that is not owned by Landcom.	**Approach 5.1** – LSW identified two pipe alignments that have manageable constraints and are amenable to the gravity drain concept that is described in this chapter. Both alignments are fully located in either the NUTRA site, a public road reserve or crown land and no unavoidable requirements for works on freehold land have been identified. The two pipe alignments are shown in preliminary design drawings provided Appendix A.
	Issue 5.2 – The proposed pipe will need to be constructed through existing residential areas and has potential to impact existing buried services.	**Approach 5.2** – LSW have reviewed existing service constraints along two pipe alignments and have not identified any unmanageable constraints associated with existing buried services. The two pipe alignments are shown in preliminary design drawings provided in Appendix A. These drawings provide information on existing services.
Constraint 6 – Elevated tailwater conditions during ocean and/or estuarine flood events	**Issue 6.1** – Water levels in the Wallis Lake Entrance Channel will occasionally exceed the normal tidal range during ocean and estuarine flooding events. This has potential to temporarily constrain the pipe capacity.	**Approach 6.1 –** Hydraulic modelling presented in Section 2.3.2, concluded that the pipe capacity will not be impacted if a 1% AEP flood event occurs with current sea level conditions. Applying projected 2100 sea level conditions, the pipe capacity would be moderately constrained (from 0.9 to 0.7 m^3/s) if a 1% AEP flood peak in the Wallis Lake Entrance Channel coincided with a peak 1% flood event in the NTURA basins. This temporary constraint will not result in a material impact to flood levels in the basins as the volume of water not conveyed due to the reduction in pipe capacity is negligible when compared to the flood storage volume in the basins (see Section 2.3.2).

2.5 Alternatives

Table 2.4 provides a description of several alternatives that were assessed during the masterplan development phase.

Table 2.4 Description of alternatives

Alternative	Description
Pump out based system	A pump out based system that discharges either to an existing drainage system, the Wallis Lake Entrance Channel via a standalone rising main or the ocean via a temporary rising main would be effective in achieving the water management objectives and would likely have lower capital costs than the proposed gravity pipe. However, a pump-out based system is not the preferred approach as it would have higher operating costs and complexities (ie associated with maintaining a large pump that is only occasionally used) and potentially lower reliability than a gravity drainage system (ie due to potential for power and mechanical failure).
Gravity drainage to the east	A gravity drain system to the east (ie directly to the ocean) would require the pipe to be constructed through the coastal foredune with an outlet within the active coastal zone. This is not considered to be appropriate given coastal hazards, impact to ecology, the potential for sand to smother the outlet when the pipe is unused and the aesthetic impacts of a pipe structure.
Gravity drainage to the west	A gravity drainage system to the west (ie into the Wallamba River) would require a greater length of pipe and would have less vertical fall than the proposed concept. It is therefore not considered to be preferable relative to the proposed concept.
Gravity drainage to the north	A gravity drainage system to the north is not possible as there is no means to outlet the pipe.

3 Design storm analysis

3.1 Overview

The IWCMS assessed flood risk using a water balance model that simulates the surface and groundwater regime by applying a long-term rainfall record for the Tuncurry area. This approach was applied as peak flooding conditions were assessed to occur due to extended periods of wet weather rather than the shorter duration intense storm events that typically lead to flooding in stormwater systems, rivers and estuaries. The water balance analysis identified a 2 ½ month rainfall sequence that occurred in 1963 as producing the highest groundwater flood levels for both existing and proposed conditions. This rainfall sequence comprised 1,464 mm over 69 days and included four embedded storms, that had 48-hour rainfall totals of between 150 to 250 mm. Analysis of the water balance results assessed this rainfall sequence as being greater than a 1% AEP event. Accordingly, this event was adopted as a pseudo 1% AEP event. The water balance model and detailed groundwater models were applied to assess this event, and the model results were used to establish the flood risk management approach for the project. Refer to IWCMS Section 5 and GWMR Section 7 for detailed information on the 1963 event and assessment approach.

Design storm analysis applying the methods and principles described in *Australian Rainfall and Runoff* (ARR) (Ball et all 2019) has been undertaken to complement the water balance analysis. The design storm analysis assesses a range of AEP storm events that have durations ranging from a few hours to 7 days. The analysis requires assumptions regarding the initial basin water and groundwater levels, which is a limitation to this approach.

The results are used to complement the water balance results and assess: flood conditions associated with 1% AEP (1 in 100 year), 0.2% AEP (1 in 500 year) and 0.05% AEP (1 in 2,000 year) events; the consequence of a blockage to the gravity pipe; and sensitivity to antecedent conditions. Analysis of the PMF is also provided.

This chapter documents the design storm analysis. Section 3.2 describes the model approach and assumptions, while model results are provided in Section 3.3.

Chapter 4 describes an updated flood risk management approach that considers the results from the water balance, detailed groundwater model and design storm analysis.

3.2 Model description

3.2.1 Approach

Design storms and the PMF were simulated using the basin routing function in XP-RAFTS. The model simulates inflows into the basins from contributing catchment areas, storage in the basin and above-basin areas and outflows from the basin in the gravity pipe. Table 3.1 describes the proposed assessment approach, some aspects are described in further detail after the table.

Table 3.1 Design storm and PMF assessment approach

Aspect	Proposed assumptions/approach	Justification
Rainfall	• Design storm analysis – 1%, 0.2% and 0.05% AEP events were simulated using ARR 2019 methods. Lower magnitude events were not assessed as they will be fully contained within the basins. Further details are provided in Section 3.2.2. • Probable Maximum Precipitation (PMP) – short and long duration events were assessed using ARR 2019 methods (see Section 3.2.2).	Standard industry approach.

Table 3.1 Design storm and PMF assessment approach

Aspect	Proposed assumptions/approach	Justification
Hydrology	The model considers all impervious and pervious areas within Zones D1, D2 and D4 (refer to the IWCMS for a description of water management zones). Runoff from Zone D3 was not included as it is an infiltration zone, and any surplus overland flow will generally drain away from the basins (see IWCMS Figure 2). Catchment areas and rainfall loss assumptions are described in Section 3.2.3.	Standard approach
Antecedent conditions (initial basin and groundwater levels)	**Design storm analysis** – antecedent conditions equivalent to 50th percentile conditions (ie median conditions) were applied. The median basin and groundwater levels are 1.8 m AHD (see IWCMS Plate 5-6). It is noted that the consequence of a design storm occurring with more conservative antecedent conditions is also assessed as a sensitivity scenario (see Section 3.3.2)	Model results are sensitive to antecedent conditions. Applying conservative antecedent conditions will effectively increase the AEP of the design event. The joint probability of intense rainfall occurring when basin and groundwater levels are elevated is assessed comprehensively in the IWCMS using a continuous simulation water balance approach.
	PMF analysis – antecedent conditions equivalent to 'minor flood conditions' were applied – the proposed antecedent basin and groundwater level is 3 m AHD. This is equivalent to 98th percentile conditions (see IWCMS Plate 5-6).	Conservative antecedent conditions are appropriate for a PMF simulation as the objective of the simulation is to establish the worst-case flood conditions.
Basin storage	The level storage curve provided in GWMR Plate 4-7 has been applied. Further details are provided in Section 3.2.4.	Based on the conceptual landform developed as part of the masterplan
Gravity pipe	The gravity pipe will drain water from the basins when the water level exceeds 3 m AHD. The rating curve provided in Figure 2.5 was applied to the design storm analysis. Refer to Chapter 2 for further information on the gravity pipe and hydraulic analysis. It is noted that the consequence of partial and full pipe blockages are assessed as sensitivity scenarios (see Section 3.3.2).	Standard approach
Losses to groundwater	No losses from the basin to the adjoining groundwater system were applied as these losses would be minor in comparison to the inflows that would occur during short duration intense storm events.	Conservative approach

3.2.2 Rainfall inputs

i Design storms

The 1%, 0.2%, and 0.05% AEP events were simulated using ARR 2019 methods. Lower magnitude events were not assessed as they will be fully contained within the basins (see model results in Section 3.3.1). The following approach was applied to establishing design rainfall inputs:

1. For each AEP a range of durations was assessed. For each duration the 10 ensemble storms were simulated and the storm that produced the 5th highest basin level (ie the result above median) was selected.

2. The critical duration was selected based on the highest simulated basin level.

ii Probable maximum precipitation

The NTURA is located within the GSAM – GTSMR Coastal Transition Zone, hence the following methods apply to calculating the PMP:

- The Generalised Short Duration Method (GSDM) applies to short duration events (≤6 hours).
- Both the Generalised South-eastern Australia Method (GSAM) and Generalised Tropical Storm Method Revised (GTSMR) apply to calculating longer duration (24 to 120 hours) events, with the method producing the highest rainfall adopted.

The PMP for storm durations between 1 and 120 hours was calculated using the above methods. Calculation sheets are provided in Appendix B.

3.2.3 Hydrology assumptions

Table 3.2 describes the catchment area and rainfall loss assumptions. The rainfall loss assumptions for pervious areas account for initial losses due to infiltration into the groundwater system. For most events, the initial losses will not be exceeded due to the very high infiltration rates and the significant storage provided in the aquifer. However, during long duration (ie 5 to 7 day events) and extreme rainfall events such as a 0.05% AEP or PMP event, runoff from pervious areas could occur:

- from the golf course and other areas within the above 1% AEP flood storage area (see Section 3.2.4) when these areas become inundated;
- if rainfall intensities exceed the near-surface infiltration rates, which were estimated to range between 70 and 1,700 mm/hr (see GWMR Section 2.4); and
- if groundwater rises to the surface, depleting aquifer storage.

Initial and continuing losses for pervious areas were calculated to capture the above mechanisms for the antecedent groundwater levels established in Table 3.1. The logic applied to each loss rate is described in the justification column and table notes.

Table 3.2 Hydrology assumptions

Land-surface	Area	Rainfall loss assumptions	Justification
Zone D1 – Golf course and open space (100% pervious surfaces)	68.6 ha	**Median conditions (design storms)** • Initial loss – 408 mm • Continuing loss – 1 mm/hr **Minor flood conditions (PMF)[4]** • Initial loss – 204 mm • Continuing loss – 1 mm/hr	The initial loss accounts for groundwater storage and is calculated as follows: *(4.2 m AHD[1] – antecedent groundwater level[1]) x the specific yield[2] of 170 mm/m* When the initial loss is exceeded, the model will assume that runoff from pervious areas will occur. This will account for direct rainfall onto the golf course if it is flooded during a > 1% AEP event. The continuing loss accounts for some groundwater flow out of the development area that would slowly replenish available groundwater storage.

Table 3.2 Hydrology assumptions

Land-surface	Area	Rainfall loss assumptions	Justification
Zone D2 – Water management basins	18.1 ha	**Open water (90% of area)** • Initial loss – 0 mm • Continuing loss – 0 mm/hr **Batters (10% of area)** • Zone D1 approach was applied	No losses due to direct rain onto an open waterbody.
Zone D4 – Piped drainage zone			
– Roof area	40.9 ha	• Initial loss – 10 mm • Continuing loss – 0 mm/hr	Initial loss accounts for some capture in rainwater tanks, assumes tanks are half full.
– Road area	29.2 ha	• Initial loss – 1.5 mm • Continuing loss – 0 mm/hr	Standard rates
– Pervious area	42.2 ha	**Median conditions (design storms)** • Initial loss – 1,088 mm • Continuing loss – 1 mm/hr **Minor flood conditions (PMF < 6 hour duration)**[3] • Initial loss – 0 mm • Continuing loss – 70 mm/hr **Minor flood conditions (PMF > 6 hour duration)**[4] • Initial loss – 680 mm • Continuing loss – 1 mm/hr	As per Zone D1, but the average surface level is increased to 5 m AHD and the initial loss is factored to account for impervious area runoff that will be conveyed to the basins (ie infiltration will only occur from pervious areas (approximately 40% of the surface area in Zone D4)).
Zone D4 (Total)	112.3 ha		
Zone D1, D2 and D4 total	199 ha		

Notes: 1. 4.2 m AHD is a conservative estimate of average pervious surface levels in Zones D1. Refer to Table 3.1 for proposed antecedent groundwater levels.
2. Specific yield is a term used to describe an aquifers ability to store and release water. A specific yield of 0.17 (or 170 mm/per m of aquifer) was established in the GWMR. From a flood modelling perspective an aquifer with a specific yield of 0.17 can store 170 mm of infiltrated rainfall for every 1 m of water table rise.
3. For short duration PMF events, rainfall intensities may exceed the near-surface infiltration rates. Hence, pervious area losses were accounted for using a continuing loss rather than an initial loss. The continuing loss rate of 70 mm/hr was selected based on the results from infiltration test site IT6 that measured the infiltration rate on a golf fairway (see GRMR Section 2.4). This approach was not applied to Zone D1 as any infiltration excess would temporarily pond on the golf course instead of draining to the basin.
4. For longer duration events, the rainfall losses are constrained by the available storage in the aquifer rather than the near-surface infiltration rate.

3.2.4 Flood storage

Flood storage will be provided within the basin (above the ambient basin level) and on low lying land that adjoins the basin, this primarily comprises the golf course. A storage level curve for the development area was established using conceptual earthworks modelling (see IWCMS Chapter 7) and is provided in IWCMS Plate 5-4 (up to 4.0 m AHD only) and GWMR plate 4-7 (for a full range of elevations). Associated design contours are provided in IWCMS Figure 2. The storage level curve was applied to design storm modelling and is reproduced in Figure 3.1. Note the curve was adjusted to show flood storage (ie storage above 1.8 m AHD) only. The 1% AEP basin and freeboard levels and associated storage volumes are provided for context. The terms basin storage and freeboard storage are used when discussing the results (see Section 3.3).

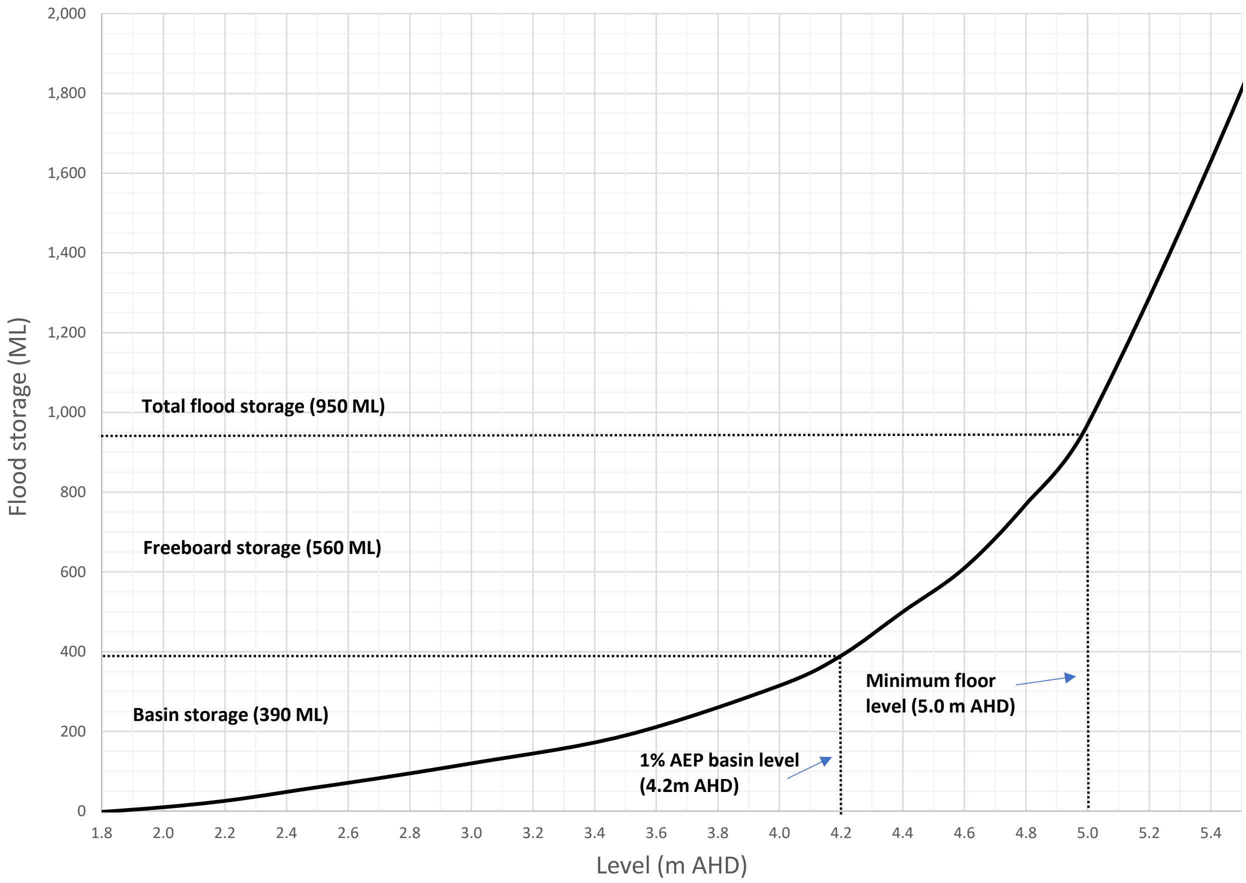

Figure 3.1 Flood storage level curve

Source: the storage level curve was sourced from GWMR Plate 4-7 but was adjusted for flood storage only (ie storage below 1.8 m AHD was removed)

3.3 Model results

This section presents the results from the simulation of the:

- 1%, 0.2%, and 0.05% AEP events (Section 3.3.1);
- sensitivity scenarios (Section 3.3.2); and
- PMF scenarios (Section 3.3.3).

3.3.1 Design storm analysis

i Results

The 1%, 0.2%, and 0.05% AEP events were simulated by applying the methods and assumptions described in Section 3.2. The following results are provided:

- Table 3.3 presents the peak simulated basin water levels for the 1%, 0.2%, and 0.05% AEP events for storm durations ranging from 12 hours to 7 days. The peak levels associated with the 10 ensemble storms are provided in Appendix C.
- Basin water level hydrographs for the 48-hour (2-day) and 144-hour (6-day) duration storm events are provided in Figure 3.2 to Figure 3.4 for the 1%, 0.2%, and 0.05% AEP events respectively. The basin storage level and minimum floor levels (established in Chapter 4) are provided for context.

The results are discussed following the tables and figures.

Table 3.3 Design storm results: peak basin water levels

	Peak simulated basin water level (m AHD)		
Storm duration	**1% AEP (1 in 100 yr)**	**0.2% AEP (1 in 500 yr)**	**0.05% AEP (1 in 2,000 yr)**
12 hr	3.51	3.73	3.94
24 hr	3.69	3.94	4.19
36 hr	3.79	4.06	4.35
48 hr (2 day)	3.84	4.19	4.47
72 hr (3 day)	3.87	4.25	4.56
96 hr (4 day)	3.89	4.29	4.61
120 hr (5 day)	4.01	4.34	4.63
144 hr (6 day)	**4.04**	**4.39**	**4.70**
168 hr (7 day)	4.03	4.37	4.67

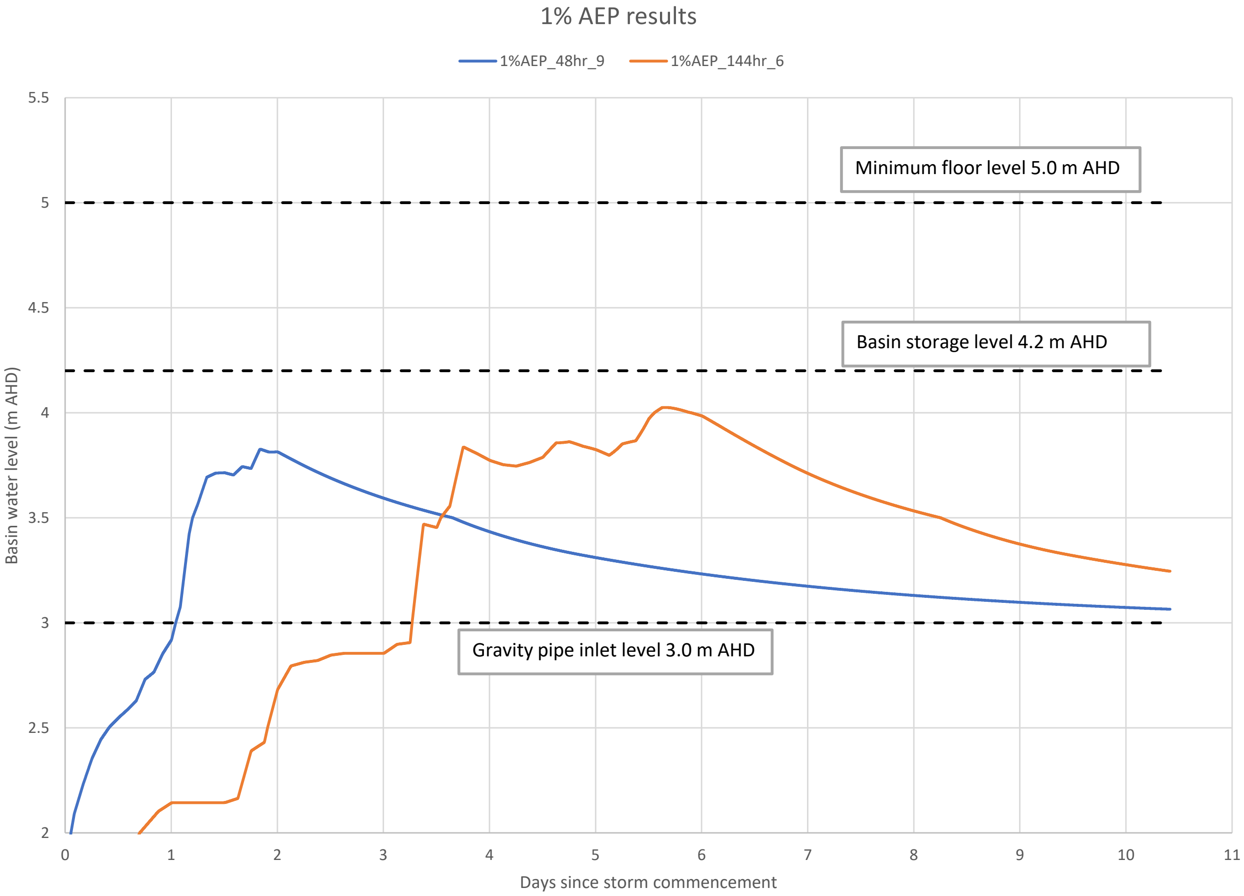

Figure 3.2 Basin water level hydrographs – 1% AEP event

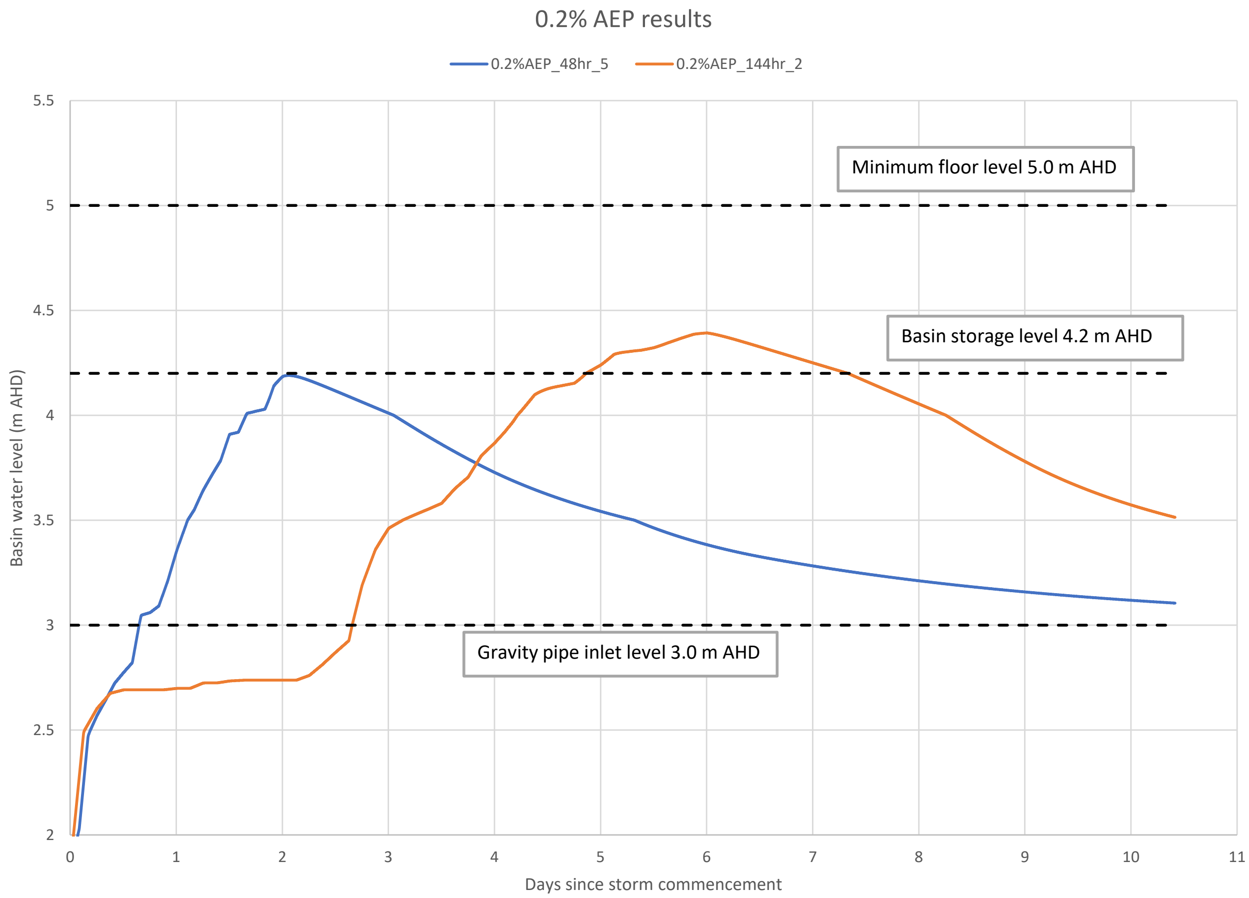

Figure 3.3 Basin water level hydrographs – 0.2% AEP event

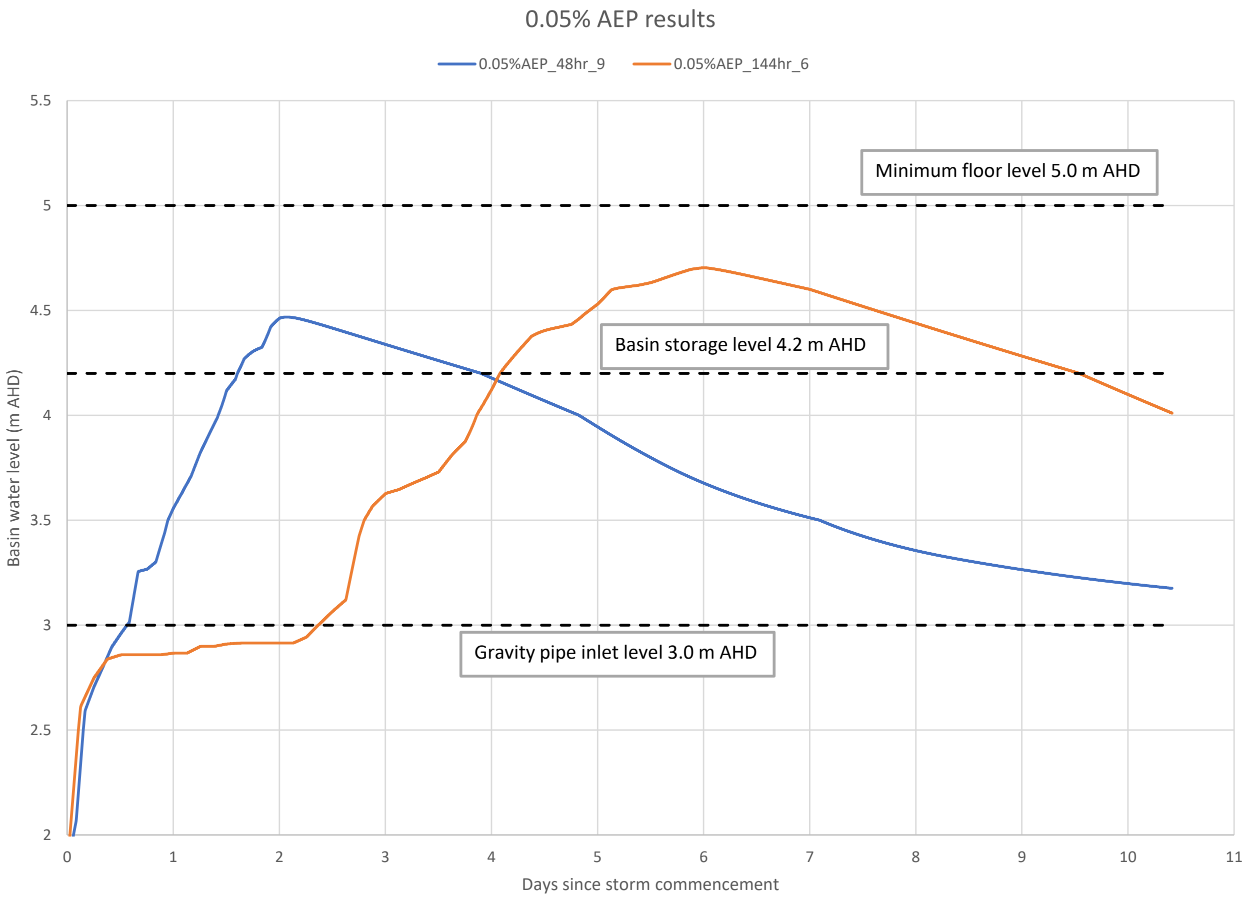

Figure 3.4 Basin water level hydrographs – 0.05% AEP event

ii Discussion

The design storm analysis assessed the basin flooding regime for events that have durations between 12 hours and 7-days. For the 1% AEP (1 in 100 year) storm events, the peak basin water levels ranged from 3.51 to 4.04 m AHD, with the highest simulated level occurring from the 6-day duration event (see Table 3.3). This analysis indicates that flooding during design rainfall 1% AEP events will be fully contained within the basin storage (see Section 3.2.4).

Analysis of the 0.2% AEP (1 in 500 year) event concluded that flooding from storm durations of up to 48 hours would be fully contained within the basin storage. Longer duration events would produce peak flood levels of up to 4.39 m AHD, resulting in shallow (<0.2 m) flooding of the golf course and perimeter roads around the basins. This flooding would last for approximately 2 days (see Figure 3.3).

Analysis of the 0.05% AEP (1 in 2,000 year) event concluded that flooding from storm durations of up to 24 hours would be fully contained with the basin storage. Longer duration events would produce, peak flood levels of up to 4.70 m AHD, which is 0.3 m below the proposed minimum habitable floor level (it is noted that the simulated level is a still level and higher levels may occur due to small waves). Flooding above the basin storage level would last for approximately 5 days (see Figure 3.4).

3.3.2 Sensitivity scenarios

i Scenarios

Analysis was undertaken to estimate peak basin flood levels for a range of scenarios that include partial and full blockage of the gravity drain and conservative antecedent conditions. The following scenarios were assessed for the 1% AEP event:

- **Scenario 1** – applies a 50% reduction in the gravity drain capacity.
- **Scenario 2** – applies a 100% reduction in the gravity drain capacity (ie assumes no flow through the pipe).
- **Scenario 3** – applies the conservative 'minor flood' antecedent conditions that were established in Section 3.2 for use in the long duration PMF simulation. These conservative antecedent conditions apply a higher initial basin water level and lower rainfall losses for pervious areas.
- **Scenario 4** – applies the conservative antecedent conditions combined with a 50% reduction in the gravity drain capacity.
- **Scenario 5** – applies the conservative antecedent conditions combined with a 100% reduction in the gravity drain capacity (ie assumes no flow through the pipe).

ii Results

The peak simulated basin water levels for each sensitivity scenario are provided in Table 3.4. For each scenario, the peak water level for storm durations up to 48 hours (2 days) and the highest level from all durations is provided. Basin water level hydrographs for the 48 hour (2 day) and the governing duration storm events are provided in Appendix C. Results are discussed following the table.

Table 3.4 Sensitivity scenario results: peak basin water levels

	Peak simulated basin water level (m AHD) for the 1% AEP event	
	Storm durations up to 2-days	**Highest level - all storm durations**
Scenario 1 50% reduction in gravity drain capacity	3.92	4.22 (7-day storm)
Scenario 2 100% reduction in gravity drain capacity	4.01	4.43 (7-day storm)
Scenario 3 Conservative initial conditions[1]	4.28	4.33 (6-day storm)
Scenario 4 Conservative initial conditions[1] plus 50% reduction in gravity drain capacity	4.37	4.53 (6-day storm)
Scenario 5 Conservative initial conditions[1] plus 100% reduction in gravity drain capacity	4.48	4.78 (7-day storm)

Notes: 1. Applies the minor flood antecedent conditions established in Section 3.2.

iii Discussion

a Gravity drain blockage scenarios

Scenarios 1 and 2 assess the implications of a partial and full reduction in the gravity drain capacity. The results indicate that for both scenarios peak flood levels would not exceed the basin storage level if a partial or full blockage of the gravity drain occurred during a 2-day (or shorter duration) storm event. For longer duration events (ie the 7-day event), the basin storage level would be exceeded by 0.02 m for Scenario 1 (50% capacity reduction) and 0.23 m for Scenario 2 (a 100% capacity reduction). The peak Scenario 2 level is 0.57 m below the proposed minimum floor level. As most blockage issues could be resolved within 24 hours via maintenance, the likelihood of a full blockage occurring for 7 days is low.

It is also noted that this analysis may appear to understate the importance of the gravity drain. The drain's primary purpose is to manage the accumulation of water during prolonged wet weather periods, such as the 1963 rainfall sequence. Water balance modelling of this sequence estimated that the drain will remove 974 ML of water from the basins over a period of approximately 69 days (see GWMR Plate 7-8). If this water was not removed, the basin water level would likely exceed 5 m AHD and could take several months to recede (via groundwater flow).

b Conservative antecedent conditions scenario

Scenario 3 simulates the implications of a design 1% AEP event occurring with conservative 'minor flood' antecedent conditions. The applied conditions assume an initial basin and typical groundwater level of 3 m AHD. Such conditions would only occur following either a significant rainfall event or an extended wet period. Water balance model results indicate that these levels are representative of 98th percentile conditions (ie they occur only 2% of the time) (see IWCMS Plate 5-6). Accordingly, a design 1% AEP rainfall event occurring with 'minor flood' antecedent conditions would occur less frequently then a 1% AEP rainfall event. It is noted that the joint probability of intense rainfall occurring when basin and groundwater levels are elevated is assessed comprehensively in the IWCMS using a continuous simulation water balance approach.

The Scenario 3 results indicate that if a design 1% AEP rainfall event was to occur with 'minor flood' antecedent conditions, the peak water level would exceed the basin storage level by 0.13 m, but would be 0.67 m below the proposed minimum floor level.

c Scenarios 4 and 5

Scenarios 4 and 5 consider the implications of a partial or full blockage to the gravity drain combined with conservative antecedent conditions. The simulated peak flood levels for both Scenarios are within the freeboard zone, with no over floor flooding predicted.

3.3.3 PMF

i Scenarios

The PMF was simulated by applying the methods and assumptions described in Section 3.2. The following scenarios were simulated:

- **Short duration events** – applies PMP storm durations between 1 and 6 hours that were calculated using the GSDM.
- **Long duration events** – applies PMP storm durations between 1 and 5 days that were calculated using the GTSMR. PMP storms calculated using the GSAM were not assessed as they have lower rainfall totals than the same storm calculated using the GTSMR (see Appendix B).

ii Results

For each scenario, the total rainfall, peak basin level and maximum rate of water level rise (above the basin storage and minimum floor levels) is provided in Table 3.5. Basin water level hydrographs for the 6-hour and 5-day duration events are provided in Figure 3.5. Results are discussed following the table and figures.

Table 3.5 PMF results: peak level and rate of rise

Duration	Total rainfall	Peak basin level	Maximum rate of rise[1]	
			Above basin storage level (4.2 m AHD)	Above minimum floor level (5.0 m AHD)
Short duration events (PMP calculated using the GSDM)				
1 hr	360 mm	4.6 m AHD	0.7 m/hr	N/A
1.5 hr	470 mm	4.9 m AHD	0.9 m/hr	N/A
2 hr	550 mm	5.0 m AHD	0.8 m/hr	N/A
2.5 hr	600 mm	5.1 m AHD	0.8 m/hr	0.1 m/hr
3 hr	660 mm	5.1 m AHD	0.7 m/hr	0.1 m/hr
4 hr	760 mm	5.2 m AHD	0.6 m/hr	0.1 m/hr
5 hr	830 mm	5.3 m AHD	0.6 m/hr	0.1 m/hr
6 hr	880 mm	5.3 m AHD	0.5 m/hr	0.1 m/hr
Long duration events (PMP calculated using the GSDM)				
24 hours (1 day)	810 mm	5.1 m AHD	0.2 m/hr	0.1 m/hr
36 hours (1 ½ days)	990 mm	5.3 m AHD	0.1 m/hr	0.1 m/hr
48 hours (2 days)	1,160 mm	5.5 m AHD	0.1 m/hr	0.1 m/hr
72 hours (3 days)	1,460 mm	5.7 m AHD	0.2 m/hr	0.1 m/hr
96 hours (4 days)	1,640 mm	5.9 m AHD	0.1 m/hr	0.1 m/hr
120 hours (5 days)	1,720 mm	5.9 m AHD	0.1 m/hr	0.1 m/hr

Notes: 1. Calculated as the maximum average rate of rise over a 15-minute period and rounded to one decimal place.

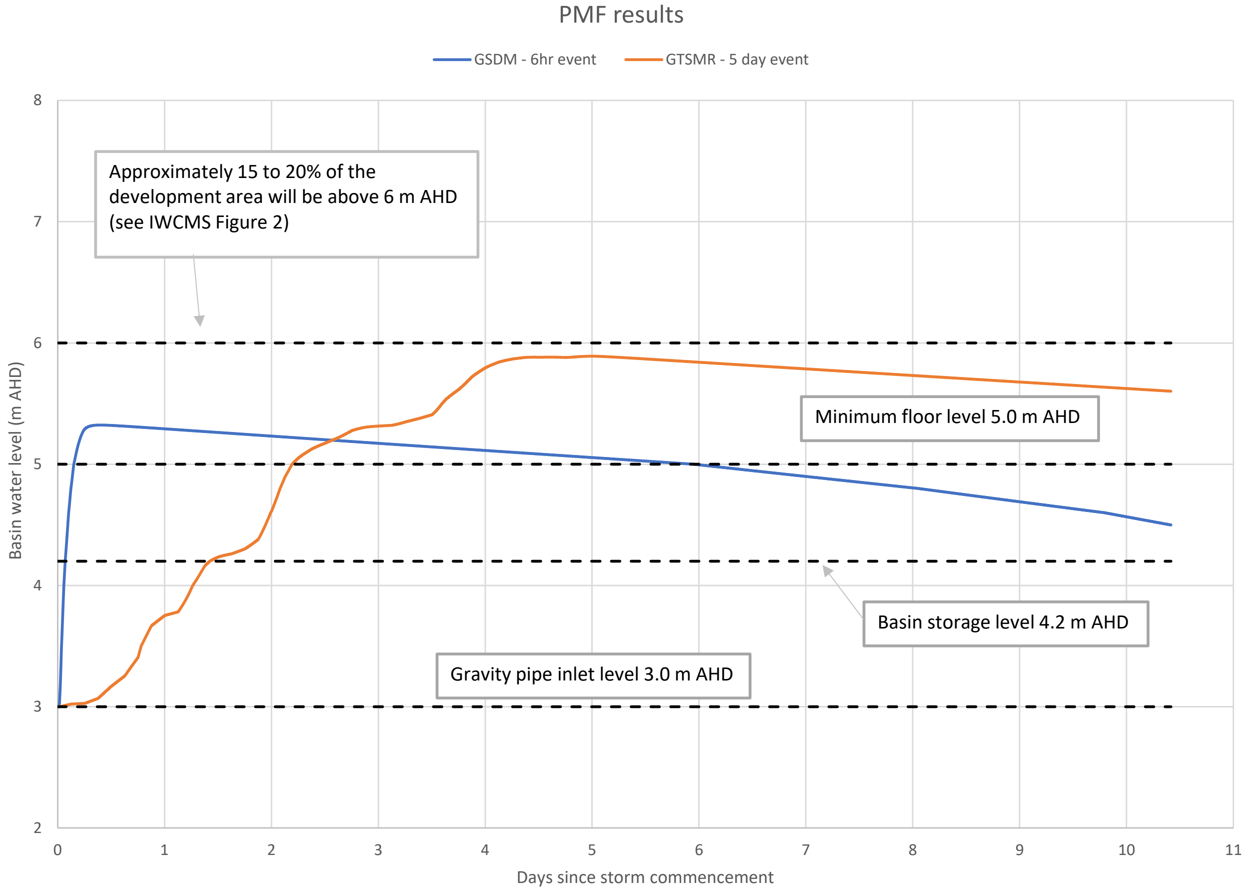

Figure 3.5 Basin water level hydrographs – PMF

iii Discussion

The PMF simulation results indicate that for a short duration event (ie a 6-hour duration PMP event calculated using the GSDM), the water level in the basins would rise from 3.0 to 5.3 m AHD over a 6-hour period. This would result in inundation of the golf course and low-lying development areas. The maximum rate of rise (above the basin storge level) would be approximately 0.4 to 0.5 m/hour. A peak flood level of 5.3 m AHD would exceed the minimum floor level by 0.3 m, resulting in shallow inundation of lower lying dwellings within the development area. Flood waters would take more than a week to recced below the minimum floor level. A shelter in place strategy during the storm followed by evacuation of impacted dwellings would be an appropriate risk management approach.

The PMF simulation of a longer duration event (ie a five-day duration event calculated using the GTSMR) indicates that floodwaters would slowly rise (at approximately 1 m/day, with short term rises of 0.1 m/hr during intense rainfall bursts) and ultimately reach 5.9 m AHD, though it is possible that some water may 'spill' from the development area via low points in the developed landform. If such an event were to occur, evacuation of most dwellings would be required. As noted in Figure 3.5, approximately 15 to 20% of the development area would be above 6 m AHD.

In summary, a PMF event would be of major concern to residents within NTURA as well as the greater Foster Tuncurry region. However, due to the slow rate of rise of flood waters, a PMF would not present any material unmanageable risk to life.

3.3.4 Summary

The design storm analysis has estimated a peak 1% AEP basin level of 4.04 m AHD, which is higher than the level documented in the IWCMS (3.9 m AHD) that was established using the water balance model. This higher level has been applied to the revised flood risk management approach that is described in Chapter 4.

The results from the 0.2% and 0.05% AEP events and sensitivity analysis demonstrates that the flood storage within the freeboard zone (ie 4.2 to 5.0 m AHD) provides significant contingency for events and/or circumstances that are beyond those applied to the design event simulations. For example, the analysis concluded that peak basin flood levels would not exceed the proposed minimum floor level of 5.0 m AHD: during a 0.05% AEP (or 1 in 2000 year) design storm event; or if a 1% AEP event occurred following an extended wet period and the gravity drain was fully blocked.

In summary, the design storm analysis demonstrates that the proposed water management system can effectively manage shorter duration intense storm events as well as the longer duration (ie several months) extended wet periods that were the focus of the detailed modelling documented in the IWCMS and GWMR.

4 Flood risk management

4.1 Overview

The flood risk management strategy has been updated to incorporate the results from the water balance model, detailed groundwater model and design storm analysis. This chapter describes the updated approach (Section 4.2) and explains why no off-site flood impacts are expected (Section 4.3).

4.2 Flood risk management approach

This section describes the proposed flood planning levels and risk management measures.

4.2.1 Flood planning levels

Table 4.1 describes the proposed flood planning levels. This information supersedes the flood planning levels provided in the IWCMS.

Table 4.1 Proposed flood planning levels

Flood planning level	Proposed levels	Rationale
1% AEP groundwater flood levels	Groundwater levels provided in IWCMS Plate 5-11 It is noted that peak groundwater levels will vary locally (ie due to mounding near infiltration systems). Hence, appropriate freeboards will be established at Concept Design (see Chapter 5) using a risk assessment framework.	• Calculated using the detailed groundwater model[1] based on the simulation of the 1963 event. • Incorporates 2100 sea level rise conditions
1% AEP basin water level	• Golf course basins[2] – 4.2 m AHD • Northern basins[2] – 4.4 m AHD	**Golf course basins** The 1% AEP basin level is the minimum surface level (see IWCMS Figure 4) and is conservatively higher than the peak 1% AEP level of: • 4.04 m AHD that was calculated by the design storm analysis (see Section 3.3.1); and • 3.9 m AHD that was calculated by the water balance model (see IWCMS). **Northern basins** The level is increased by 0.2 m to allow for some head loss as water flows from these basins to the gravity drain inlet that will be in the southern most basin.

Table 4.1 Proposed flood planning levels

Flood planning level	Proposed levels	Rationale
Minimum habitable floor levels	**Zone D4 (piped drainage zone)** The greater of: • 0.8 m above the 1% AEP basin level • 0.5 m above the adjoining road level[4]	This approach applies a 0.8 m freeboard to the 1% AEP basin level. As described in Chapter 3, the flood storage provided in the freeboard zone provides significant contingency. The 0.5 m freeboard to the adjoining road level will mitigate risks associated with groundwater flooding as: • roads will be at or above the 1% AEP groundwater levels; • the roads will have subsurface drainage (see Measure 2.1 in Table 4.2 for further information); and • any groundwater that intercepts the surface will drain freely to the stormwater system within the road reserve (see Measure 2.2 in Table 4.2 for further information).
	Zone D3 (infiltration zone) • Provisional level[3] of 1.0 m above the 1% AEP groundwater level or 5 m AHD (whichever is higher)	A provisional 1 m freeboard to the 1% AEP groundwater level is applied as there is potential for localised mounding of the watertable near infiltration areas. This freeboard will be reviewed at Detailed Design once the infiltration system design is known. The minimum floor level of 5 m AHD will provide the same mitigation as Zone D4 for dwellings located near the basins.
Minimum road levels	**Zone D4 (piped drainage zone)** The greater of: • the 1% AEP basin level; or • the 1% AEP groundwater level	This approach will ensure that roads are trafficable during a 1% AEP event.
	Zone D3 (infiltration zone) • Provisional level[3] of 0.5 m above the 1% AEP groundwater level	A provisional 0.5 m freeboard to the 1% AEP groundwater level is applied as there is potential for localised mounding of the watertable near infiltration areas. This freeboard will be reviewed at Detailed Design once the infiltration system design is known.
PMF level	5.9 m AHD	This is the peak PMF level that was calculated using the methods described in Chapter 3.

Notes: 1. Refers to the three-dimensional groundwater model that was developed using the MODFLOW-SURFACT modelling platform. Refer to GWMR Section 5 for a description of the detailed groundwater model.
2. The golf course basins refer to the basins that adjoin the golf course. The northern basins refer to the northern most basin and the two finger drains that extend to the north and north-west from this basin. These basins are noted in IWCMS Figure 1 and 2.
3. This level is provisional as it will be reviewed and potentially altered when preparing a Development Control Plan or at detailed design.
4. Refers to the lowest road level that adjoins the lot.

4.2.2 Proposed measures

Table 4.2 describes proposed flood risk management measures. This information supersedes the measures provided in the IWCMS. It is noted that these measures are provisional as they will be reviewed and potentially altered during future design stages.

Table 4.2 Proposed flood risk management measures

Provisional[1] Measure	Rationale
1 - General	
Measure 1.1 – The water management system will be designed and constructed generally in accordance with the concepts described in the IWCMS and this addendum report.	To ensure the system functions in a manner that is consistent with the functionality assumed when preparing the IWCMS and addendum reports and associated modelling.
Measure 1.2 – Roads and other buried infrastructure will be designed and constructed to be resilient to groundwater inundation.	To minimise the risk of infrastructure damage due to groundwater flooding.
Measures 1.3 – The potential for a near surface watertable will be considered in the design and construction of dwellings and other structures.	To ensure periods of elevated groundwater levels do not cause structural damage to dwellings or indirect damage (ie due to rising damp).
2 - Zone D4 (piped drainage zone)	
Measure 2.1 – Subsurface drainage will be provided under road bases in the piped drainage zone. The drainage will outlet into the piped drainage system.	This measure was recommended in the IWCMS to manage localised groundwater flooding in development areas.
Measure 2.2 – All lots will be free draining to either a road gutter or a surface drain. The floor levels of dwellings will be located sufficiently above drain levels to ensure that the dwellings will not be inundated by overland flows that could occur from stormwater runoff or if the watertable intercepts the surface.	To ensure that dwellings will not be inundated by overland flows that could occur from stormwater runoff or if the watertable intercepts the surface.
3 - Zone D3 (infiltration zone)	
Measure 3.1 – All lots will be free draining to either a road gutter or a surface drain. The floor levels of dwellings will be located sufficiently above drain levels to ensure that the dwellings will not be inundated by overland flows that could occur from stormwater runoff; blockage of an infiltration system; or if the watertable intercepts the surface.	To ensure that dwellings will not be inundated by overland flows that could occur from stormwater runoff, blockage of an infiltration system or if the watertable intercepts the surface.

Notes: 1. The measures are provisional as they will be reviewed and potentially altered when preparing a Development Control Plan or at detailed design.

4.3 Flood impacts on adjoining land

The project is not expected to result in impacts to groundwater or surface water flooding on land adjoining the site or the golf course, which will be partially reconfigured as part of the project. Table 4.3 provides a summary of potential impact mechanisms and explains why no impacts are predicted.

Table 4.3 Summary of flood impacts

Flood impact mechanism	Summary of impacts
Increases to peak groundwater flood levels	The proposed water management system will lower peak groundwater flood levels both within the site and in areas adjacent to the site. GWMR Plate 7-18 shows the predicted change in peak levels during the simulated 1963 event, which is used as a pseudo 1% AEP event. The reductions in groundwater level (relative to existing conditions) in areas adjoining the site range from 0.2 to 0.4 m to the south of the site and 0.6 to 1.0 m to the west and north of the site. In summary, no increases to groundwater flood levels are predicted within the site or on private property or crown land that adjoins the site.

Table 4.3 Summary of flood impacts

Flood impact mechanism	Summary of impacts
Impacts to surface water flooding or stormwater system capacity in Tuncurry and other surrounding areas.	**Stormwater management zones D1, D2 and D4 (Central and Western portions of the site)** All stormwater runoff and groundwater will be managed by the water management system. The system is conceptually designed to contain/store surplus runoff that will occur during intense rainfall events or prolonged periods of wet weather. When the basin levels exceed 3 m AHD, surplus water will drain to the Wallis Lake Entrance Channel in the gravity drain (see Chapter 2). No impacts to surface water flooding or the existing capacity of stormwater drainage systems in Tuncurry or other surrounding areas will occur as: • the water management system is not designed to overflow; and • the gravity drain will be standalone (ie it will not be integrated with existing drainage). **Stormwater management zone D3 (Eastern portion of the site)** The groundwater constraints are lower in the eastern portion of the development area due to proximity to the ocean. Accordingly, infiltration-based stormwater systems are proposed (see IWCMS Section 5). The stormwater systems will include surface drains that will convey runoff that exceeds the infiltration system capacity to a designated overflow location. Potential overflow locations are shown in IWCMS Figure 2. As shown in IWCMS Figure 2, stormwater runoff from a small development area located in the south-eastern portion of the site will drain to the south onto an existing sports field, where it would rapidly infiltrate. Accordingly, no impacts to the urban areas further to the south are expected.
Impacts to the Tuncurry Golf Course	The proposed water management system will reduce the frequency of groundwater levels within the development area and golf course exceeding 3 m AHD (see IWCMS Plate 5-7). As most golf greens are higher than 3 m AHD, the water management system will reduce the frequency and duration of golf greens being inundated when the groundwater table intercepts the surface. It is proposed to reconfigure some parts of the golf course as part of the project. The reconfiguration will involve establishing some new greens and will provide an opportunity to locally raise any portions of existing greens that are below 3 m AHD (if deemed necessary).

5 Design development framework

The proposed water management system described in the IWCMS (SMEC 2019) and this addendum report was developed as part of the master planning process for the project. The system conceptualisation had multiple objectives including flood risk management, water quality management, balancing cut and fill and integration with the proposed land uses. Groundwater and surface water flooding risks were identified as a key constraint in the central and western portions of the site and have been assessed using several modelling methods. The IWCMS (SMEC 2019) established four water management zones based on proposed land use and site constraints. These zones are described as Zone D1 (golf course and open space), Zone D2 (basins), Zone D3 (urban development, infiltration zone) and Zone D4 (urban development, piped drainage zone). Water management concepts for each of these zones have been established. Overall, the concepts have been developed to a sufficient level of detail to demonstrate functionality, proof of concept and establish flood planning controls (see Chapter 4).

Further design development is required for some aspects to establish the optimal design solution, development staging and finalise any planning arrangements. Accordingly, Landcom proposes that Concept Designs will be prepared for the following project elements: gravity drain, basin system and earthworks and staging. It is proposed that the Concept Designs will be finalised after rezoning approval but prior to the detailed design of development stages that will utilise the basin system. The Concept Designs will be generally in accordance with the concepts described in the rezoning proposal but will be progressed to a sufficient level of detail to enable key design decisions to be made in advance of detailed design, which will occur in stages.

The concept designs will be prepared in consultation with Council and BCD. For the gravity drain and basin system designs, Landcom will also fund external peer reviews to add both value and confidence to the process. The peer reviewer(s) will be selected in consultation with Council and will provide an independent perspective. It is expected that each design process will include a series of workshops attended by all stakeholders.

Table 5.1 provides and overview of the proposed objectives, scope, and consultation for each of the three Concept Design packages.

Table 5.1 Concept Design overview

Concept Design	Objectives	Informed by	Consultation
Gravity Drain	• Establish preferred alignment • Finalise inlet and outlet arrangements • Establish materials and construction methods • Establish any impacts to services and freehold land • Prepare a maintenance plan • Confirm hydraulic capacity	• Detailed survey (including services) • Geotechnical investigations • Civil design • Hydraulic calculations	Council, BCD, service providers (if required), independent peer reviewer
Basin systems and stormwater management	• Finalise the shape, bottom levels and the distribution of ephemeral and open water zones • Prepare conceptual civil and geotechnical designs • Finalise landscape design including planting • Finalise stormwater management approach and stormwater basin interfaces • Establish materials and construction methods • Prepare a maintenance plan • Confirm design flood levels	• Civil design • Geotechnical design • Landscape design (including planting) • Update design storm analysis (if required)	Council, BCD, independent peer reviewer
Earthworks and staging	• Update the earthworks concept to achieve cut to fill balance • Establish development staging schedule and associated progressive cut to fill balance	• Civil design	Council

6 Water regulation

6.1 Overview

This chapter addresses relevant water regulations using information sourced from the IWCMS and GWMR. It supersedes information related to water licensing and the NSW Aquifer Interference Policy (AIP) that is documented in IWCMS Section 2.3 and Appendices A and D.

6.2 Relevant regulation

The *NSW Water Management Act 2000* (WM Act) sets out the legislative requirements for the sustainable and integrated management of water sources within NSW. The principal instruments under the WM Act are management plans for: water sharing, water use, floodplain management, drainage, environmental protection, controlled activities and aquifer interference.

Plans that are relevant to the NTURA proposal are described below.

6.2.1 Water sharing plans

Water Sharing Plans (WSPs) are statutory documents that apply to one or more water sources. They define the rules for sharing and managing water resources within water source areas.

The WSPs relevant to the NTURA are:

- *Water Sharing Plan for the North Coast Coastal Sands Groundwater Sources 2016* – the Great Lakes Coastal Sands Groundwater Source applies to groundwater in the vicinity of the site.
- *Water Sharing Plan for the Lower North Coast Unregulated and Alluvial Water Sources 2009* – the Wallamba River water source applies to surface water in the vicinity of the site.

6.2.2 NSW Aquifer Interference Policy

Projects that intercept groundwater need to consider the AIP. The AIP defines the regime for protecting and managing the impacts of aquifer interference activities on NSW's water resources. The AIP requires consideration of the potential impacts of an aquifer interference activity in respect to the watertable, water pressure and water quality. Proponents must estimate the water take (including incidental take) from each water source and connected water sources. Changes to watertable, water pressure and water quality are assessed against minimal impact considerations for each water source.

The AIP defines water sources as being either 'highly productive' or 'less productive' based on levels of salinity and average available yields and by their lithological character, being one of alluvium, coastal sand, porous rock, or fractured rock, and identifies thresholds for minimal impact considerations. Based on the NSW Government's mapped areas of groundwater productivity in NSW (NOW 2012), the project area is within a 'highly productive' coastal sands water source. Applicable minimal impact considerations for the project have been reproduced in Table 6.1.

If an activity is assessed as being 'minimal impact' or the impacts are no more than the accuracy thresholds of the model, then it is defined as a 'minimal impact'. Where impacts are predicted to be 'greater than minimal impact' but additional studies show that impacts, although greater than 'minimal' do not prevent the long-term viability of the relevant water dependent asset, then the impacts will be defined as 'acceptable'. Where impacts are predicted to be 'greater than minimal impact' and the long-term viability of the water dependent asset is compromised, then the impact is subject to 'make good' provisions.

Table 6.1 Minimal impact criteria for 'highly productive' coastal sands water source

Watertable	Water pressure	Water quality
1. Less than or equal to 10% cumulative variation in the watertable, allowing for typical climatic 'post-water sharing plan' variations, 40 m from any: a) high priority groundwater dependent ecosystem; or b) high priority culturally significant site; listed in the schedule of the relevant water sharing plan. A maximum of a 2 m decline cumulatively at any water supply work. 2. If more than 10% cumulative variation in the water table, allowing for typical climatic 'post-water sharing plan' variations, 40 m from any: a) high priority groundwater dependent ecosystem; or b) high priority culturally significant site; listed in the schedule of the relevant water sharing plan then appropriate studies (including the hydrogeology, ecological condition and cultural function) will need to demonstrate to the Minister's satisfaction that the variation will not prevent the long-term viability of the dependent ecosystem or significant site. If more than a 2 m decline cumulatively at any water supply work then make good provisions should apply.	3. A cumulative pressure head decline of not more than a 2 m decline, at any water supply work. 4. If the predicted pressure head decline is greater than requirement 1 above, then appropriate studies are required to demonstrate to the Minister's satisfaction that the decline will not prevent the long-term viability of the affected water supply works unless make good provisions apply.	1. Any change in the groundwater quality should not lower the beneficial use category of the groundwater source beyond 40 m from the activity. 2. If condition 1 is not met then appropriate studies will need to demonstrate to the Minister's satisfaction that the change in groundwater quality will not prevent the long-term viability of the dependent ecosystem, significant site or affected water supply works.

6.3 Estimated water take

The proposed conditions water cycle is described in IWCMS Section 5. Figure 6.1 reproduces IWCMS Plate 5-3 (which conceptually shows the water cycle) and notes the following water take mechanisms.

1. The capture of roof runoff in rainwater tanks.
2. Net groundwater inflows into the water management basins from the adjoining coastal sands aquifer.
3. The extraction of groundwater from the coastal sands aquifer for the purposes of irrigation.
4. Groundwater inflows into the water management basins during the excavation of the basins.

For each mechanism, an assessment of water take and a description of the proposed licencing approach is provided in Table 6.2 (after Figure 6.1).

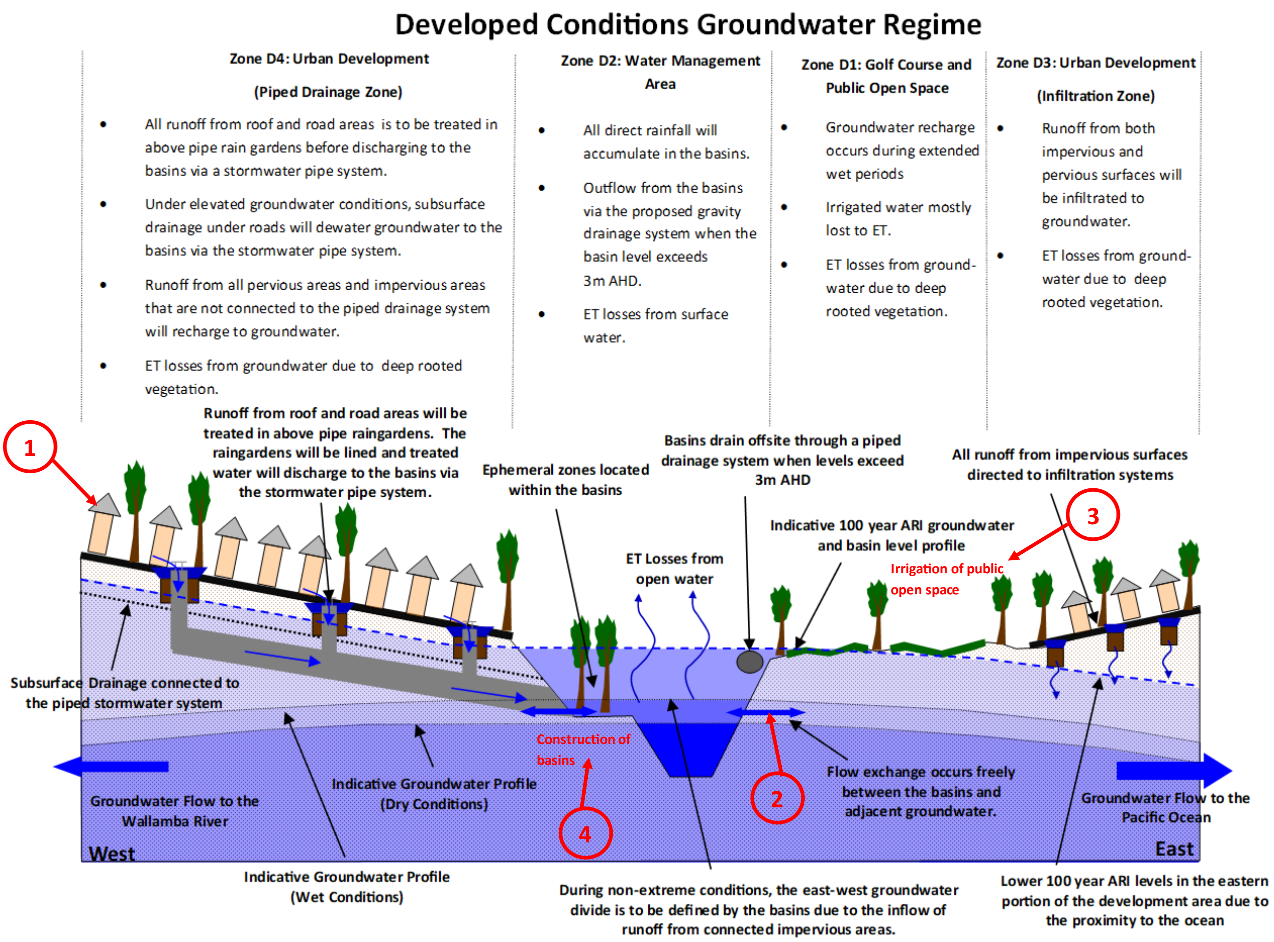

Figure 6.1 Water take mechanisms

Source: Background image from IWCMS Plate 5-3

Table 6.2 Assessment of water take and licensing approach

Water take mechanism	Assessment of water take	Licensing approach
The capture of water in rainwater tanks	The rainwater tanks are excluded works under Schedule 1, item 5 of the NSW Water Management (General) Regulation 2018. Reuse of water captured within the rainwater tank is exempt from requiring a licence under Schedule 4, item 12 of the NSW Water Management (General) Regulation 2018.	There is no requirement for a water access licence or works approval.
Net groundwater inflows into the water management basins from the adjoining coastal sands aquifer.	**Incidental groundwater take during non-flood conditions** During non-flood conditions, the water level in the basins will be higher than levels in the surrounding groundwater system as runoff from impervious areas in stormwater management Zone D4 (see IWCMS Plate 6-5) will be conveyed into the basins, along with direct rainfall. As a result, water from the basins will slowly flow to the east and west into the groundwater system (see IWCMS Plate 5-9). Accordingly, no water take is predicted.	There is no requirement for a water access licence or works approval.
	Incidental groundwater take during flood conditions Groundwater modelling results for both existing and proposed development scenarios show that during flood conditions, the east-west groundwater divide will shift to the west of the development area (see GWMR Section 8). This is due to the increase in groundwater levels resulting from high rainfall recharge. As a result, groundwater would flow into the basins from the west and out of the basins to the east, towards the ocean (see IWCMS Plate 5-11). Hence, there is potential for incidental groundwater take to occur if the groundwater inflows exceed outflows. Mass balance results presented in GWMR Plates 7-7 and 7-8 include estimates of groundwater flows from the development area (in all directions) during the 1963 flood event (which is used as a pseudo 1% AEP event). The mass balance results indicate that groundwater flows would reduce from 725 ML (existing conditions) to 706 ML (developed conditions), a 19 ML change. This provides a reasonable estimate of potential incidental groundwater take volumes during flood conditions.	Advice from NSW Government's Natural Resource Access Regulator is required to ascertain if water licences are required for groundwater take that will only occur occasionally during flood conditions.
Groundwater extraction for irrigation purposes	The IWCMS notes that there is an opportunity to extract groundwater to irrigate public open spaces. Extraction would likely occur via local spearpoint wells and would be managed by Council. A potential water take volume of 67 ML/year is provided in IWCMS Section 9 based on an assumed irrigation area. This estimate will require review once detailed landscaping plans are prepared and the need for irrigation established.	Works approvals and water access licences (from the Great Lakes Coastal Sands Groundwater Source) would be obtained by or on behalf of Council prior to any extraction occurring. The NSW Water register shows that existing WALs in this water source have a total share of 2,148 ML, well below the LTAAEL[1] of 16,000 ML, indicating there is considerable market depth.

Table 6.2 Assessment of water take and licensing approach

Water take mechanism	Assessment of water take	Licensing approach
Water take associated with the excavation of the basins	The basins are effectively excavations that would satisfy the definition of an Aquifer Interference Activity (AIA) under the *Water Management Act* 2000 and therefore require approval under that Act. As the AIA approval provisions are yet to be switched on, a Water Supply Work Approval application will need to be lodged with the regulator (either WaterNSW or NRAR). Schedule 4, Clause 7 of the *Water Management Act* 2000 makes provision for exemption from requiring Water Access Licences for minimal extractions (less than 3 ML per water year) associated with an AIA. On the basis that the watertable will not need to be dewatered to excavate the basins (ie the basins will be excavated using either an excavator or a dredging system) no water take in excess of 3ML is predicted.	Work approvals will be required. The basins are considered exempt from requiring Water Access Licences provided no dewatering is required. This will be reviewed once the construction methods are known.

Notes: 1. Long Term Annual Average Extraction Limit (LTAAEL)

6.4 Assessment against Aquifer Interference Policy

The construction of the water management basins is an aquifer interference activity as defined under the AIP. Table 6.3 provides an assessment against the minimal impact criteria established in Section 6.2.2.

The aquifer interference assessment framework step by step guide is provided in Appendix D.

Table 6.3 Assessment against minimal impact criteria

Minimal impact criteria	Assessment
Watertable 1. Less than or equal to 10% cumulative variation in the watertable, allowing for typical climatic 'post-water sharing plan' variations, 40 m from any: a) high priority groundwater dependent ecosystem; or b) high priority culturally significant site; listed in the schedule of the relevant water sharing plan. A maximum of a 2 m decline cumulatively at any water supply work. 2. If more than 10% cumulative variation in the water table, allowing for typical climatic 'post-water sharing plan' variations, 40 m from any: a) high priority groundwater dependent ecosystem; or b) high priority culturally significant site; listed in the schedule of the relevant water sharing plan then appropriate studies (including the hydrogeology, ecological condition and cultural function) will need to demonstrate to the Minister's satisfaction that the variation will not prevent the long-term viability of the dependent ecosystem or significant site. If more than a 2 m decline cumulatively at any water supply work then make good provisions should apply.	**Non-flood conditions** The project will increase the volume of water recharged into the groundwater system as impervious area runoff will recharge at a greater rate and frequency than rainfall that infiltrates into the existing landscape (see GWMR Section 3 for further information). For median conditions, recharge within the development area will increase from 33% of rainfall to 50% of rainfall (see GWMR Plate 8-1). This will result in a generally higher watertable within the development area. GWMR Plate 8-2 shows the expected change in typical groundwater levels within the project area for a range of climate conditions. These results indicate that developed conditions groundwater levels will be approximately 0.3 to 0.4 m higher than existing conditions levels at all times except for flood conditions (discussed below). GWMR Plate 8-6 shows the predicted change in groundwater levels during typical wet conditions. The results indicate that groundwater levels in areas adjoining the project area will be between 0.0 and 0.3 m higher than existing levels, with the greatest increases occurring immediately to the west of the project area. Accordingly, during non-flood conditions, the project will not result in a watertable decline at any water supply work that is located near the project. **Flood conditions** During flood conditions, the water management system will lower peak groundwater flood levels, which in some locations intercept the surface. GWMR Plate 7-18 shows the predicted change in peak levels during the simulated 1963 event, which is used as a pseudo 1% AEP event. The predicted reduction in peak groundwater levels in areas adjoining the site range from 0.2 to 0.4 m to the south of the site and 0.6 to 1.0 m to the west and north of the site. The reductions will be temporary and will only occur during flood conditions, when groundwater availability is high. **Conclusion** The project will result in minor changes to local groundwater levels and groundwater availability and therefore is considered to have a minimal impact.

Table 6.3 Assessment against minimal impact criteria

Minimal impact criteria	Assessment
Water Pressure 1. A cumulative pressure head decline of not more than a 2 m decline, at any water supply work. 2. If the predicted pressure head decline is greater than requirement 1 above, then appropriate studies are required to demonstrate to the Minister's satisfaction that the decline will not prevent the long-term viability of the affected water supply works unless make good provisions apply.	The receiving groundwater system is an unconfined sand aquifer. In an unconfined system the water pressure within the aquifer is directly correlated to the watertable level, whereby a change in water pressure will result in an equivalent change in the watertable level. As a result, the assessment of changes to watertable apply to water pressure (ie minimal impact).
Water Quality 1. Any change in the groundwater quality should not lower the beneficial use category of the groundwater source beyond 40 m from the activity. 2. If condition 1 is not met then appropriate studies will need to demonstrate to the Minister's satisfaction that the change in groundwater quality will not prevent the long-term viability of the dependent ecosystem, significant site or affected water supply works.	**Potential for changes to nutrient concentrations** IWCMS Section 6.4 describes the project's predicted impacts to groundwater quality and notes the potential for the project to increase the nitrogen and phosphorus loads entering the groundwater system. For groundwater that will flow from the development area to the west towards the Wallamba River Estuary, any increased nutrient loads are expected to be attenuated to background levels via chemical sequestration/ absorption within the aquifer and assimilation by wetlands in groundwater recharge zones near the Wallamba River (refer to IWCMS Section 6.4 for further details). During both flood and non-flood conditions, the east-west groundwater divide will be to the west of the golf course (see IWCMS Plates 5-9 and 5-11). Hence, any potentially nutrient rich groundwater from the golf course will flow to the east, into the ocean which is a less sensitive receiving water than the Wallamba River Estuary. **Potential for changes to salinity** The project will increase the volume of freshwater recharged to the aquifer and is therefore not expected to have an adverse impact on water salinity. **Human health risks** All dwellings will be serviced by a sewer and there will be no on-site wastewater treatment systems such as septic tanks that will discharge into the groundwater system (see IWCMS Section 8). Accordingly, the project will not be a source of pathogens, E coli and other pollutants associated with wastewater. **Conclusion** The project will have a minimal impact on groundwater quality as it is not expected to: • result in a change in groundwater quality that would lower the beneficial use category of the groundwater source; or • impact a groundwater dependent ecosystem or receiving surface water system such as the Wallamba River.

7 References

Ball, J., Babister, M., Nathan, R., Weeks, W., Weinmann, E., Retallick, M., & Testoni, I. (Eds.). (2019). Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia

SMEC 2014, Groundwater modelling technical report, Revision B, Appendix J of the rezoning proposal

SMEC 2019, Integrated Water Cycle Management Strategy, Revision 5 - Appendix P of the rezoning proposal

WMA 2014, Wallis Lake Foreshore (floodplain) risk Management Study and Flood Study Review

Worley Parsons 2013, Coastal processes Report: Hydrodynamic and Sediment Transport Assessment of Wallis Lake Dredging

Appendix A

Gravity drain - preliminary design drawings

MGA

LEGEND

- DENOTES STORMWATER ROUTE (OPTION 1)
- DENOTES STORMWATER ROUTE (OPTION 2)

LEGEND

CODE	DESCRIPTION	LINETYPE
ELECT	UNDERGROUND ELECTRICITY	UG
FCE	FENCE	
HYD /SV	HYDRANT / STOPVALVE	
PIT	EXISTING STORMWATER	
PP /ELP	OVERHEAD ELECTRICITY	O/H
RISNG	RISING MAIN	RM
SMH	SEWER / MANHOLE	S
STN /SSM	SURVEY CONTROL	
SW /PIT	DESIGN STORMWATER	
TEL	TELSTRA	T
TRL	TREELINE	
TR	TREE	
WATER	WATER SUPPLY	W

NOTES

1. RELATIONSHIP OF IMPROVEMENTS TO BOUNDARIES IS DIAGRAMMATIC ONLY. WHERE OFFSETS ARE CRITICAL THEY SHOULD BE CONFIRMED BY FURTHER SURVEY.
2. CONTOURS SHOWN DEPICT THE TOPOGRAPHY EXCEPT AT SPOT LEVELS SHOWN. CONTOURS DO NOT REPRESENT THE EXACT LEVEL AT ANY PARTICULAR POINT.
3. SERVICES SHOWN THEREON HAVE BEEN DETERMINED FROM VISUAL EVIDENCE ONLY. PRIOR TO ANY DEMOLITION, EXCAVATION OR CONSTRUCTION ON THE SITE THE RELEVANT AUTHORITY SHOULD BE CONTACTED TO ESTABLISH DETAILED LOCATION AND DEPTH.
4. THE INFORMATION IS ONLY TO BE USED AT A SCALE ACCURACY OF 1:1000. TREE LOCATION ARE ONLY ACCURATE TO -/-0.75m
5. TREE SPREADS ARE APPROXIMATE ONLY AND FURTHER SURVEY MAY BE REQUIRED FOR ARCHITECTURAL DESIGN.
6. BEARING AND DISTANCES ARE BY TITLE AND/OR DEED ONLY. NO BOUNDARY INVESTIGATION HAS BEEN CARRIED OUT.
7. THIS DRAWING REMAINS THE PROPERTY OF LIDBURY, SUMMERS & WHITEMAN AND IS SUBJECT TO DESIGN COPYRIGHT LAWS.

ALL CONCEPTUAL ALIGNMENT OPTIONS HAVE BEEN PREPARED BASED ON GRAVITY DRAIN CONCEPT DESCRIBED IN THE ADDENDUM REPORT TO THE IWCMS (EMM 2021). THE CONCEPTS WILL BE FURTHER DEVELOPED AT THE DETAILED DESIGN.

0 5 10 20 30 40 50

SCALE 1:1000 AT A1

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PRELIMINARY
SUBJECT TO APPROVAL, FINAL SURVEY AND DESIGN

REFER TO SHEET 2 FOR CONTINUATION

NOTE: The contractor shall check and verify all works on site (including works by others) before commencing any works. All discrepancies are to be reported to the Project Manager or Lidbury, Summers & Whiteman prior to commencing work. Do not scale dimensions from the plan. All dimensions to be confirmed by dimensions from the plan. All dimensions to be confirmed by Lidbury, Summers & Whiteman.

HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913

PLOT FILE/DWG No.:
F:\D3105D\3105LD NORTH TUNCURRY.DWG

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687

DRAWN/CHECKED:
MG

PROJECT

NORTH TUNCURRY URBAN RELEASE AREA GRAVITY DRAIN: CONCEPTUAL ALIGNMENT OPTIONS

LSW
SURVEYORS, PLANNERS & ENGINEERS
LIDBURY, SUMMERS & WHITEMAN
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EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

SCALES
HORIZONTAL: 1:1000
VERTICAL: N/A
SHEET SIZE: A1

FIELD SHEETS
Date of survey:

COUNCIL: MID-COAST
DA NUMBER: XXX

DATE: 18/02/21

SHEET 1 OF 8

FILE No.: 3105LD (COMP.JOB 3105DP)

ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE

REFER TO SHEET 1 FOR CONTINUATION

LEGEND

- - DENOTES STORMWATER ROUTE (OPTION 1)
- - DENOTES STORMWATER ROUTE (OPTION 2)

LEGEND

CODE	DESCRIPTION	LINETYPE
ELECT	UNDERGROUND ELECTRICITY	UG
FCE	FENCE	
HYD /SV	HYDRANT / STOPVALVE	
PIT	EXISTING STORMWATER	
PP /ELP	OVERHEAD ELECTRICITY	O/H
RISNG	RISING MAIN	RM
SMH	SEWER / MANHOLE	S
STN /SSM	SURVEY CONTROL	
SW /PIT	DESIGN STORMWATER	
TEL	TELSTRA	T
TRL	TREELINE	
TR	TREE	
WATER	WATER SUPPLY	W

NOTES

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5. TREE SPREADS ARE APPROXIMATE ONLY AND FURTHER SURVEY MAY BE REQUIRED FOR ARCHITECTURAL DESIGN.
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7. THIS DRAWING REMAINS THE PROPERTY OF LIDBURY, SUMMERS & WHITEMAN AND IS SUBJECT TO DESIGN COPYRIGHT LAWS.

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0 5 10 20 30 40 50
SCALE 1:1000 AT A1

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PRELIMINARY
SUBJECT TO APPROVAL, FINAL SURVEY AND DESIGN

REFER TO SHEET 3 FOR CONTINUATION

NOTE: The contractor shall check and verify all works on site (including works by others) before commencing any works. All discrepancies are to be reported to the Project Manager or Lidbury, Summers & Whiteman prior to commencing work. Do not scale dimensions from the plan. All dimensions to be confirmed by dimensions from the plan. All dimensions to be confirmed by Lidbury, Summers & Whiteman.

HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913

PLOT FILE/DWG No.:
F:\D3105D\3105LD NORTH TUNCURRY.DWG

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687

DRAWN/CHECKED:
MG

PROJECT
NORTH TUNCURRY URBAN RELEASE AREA GRAVITY DRAIN: CONCEPTUAL ALIGNMENT OPTIONS

LSW
SURVEYORS, PLANNERS & ENGINEERS
LIDBURY, SUMMERS & WHITEMAN
1st FLOOR, 3 WHARF ST. FORSTER 2428
PO BOX 510 FORSTER NSW 2428
PH: (02) 6554 7988
EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

SCALES
HORIZONTAL: 1:1000
VERTICAL: N/A
SHEET SIZE: A1

FIELD SHEETS
Date of survey:

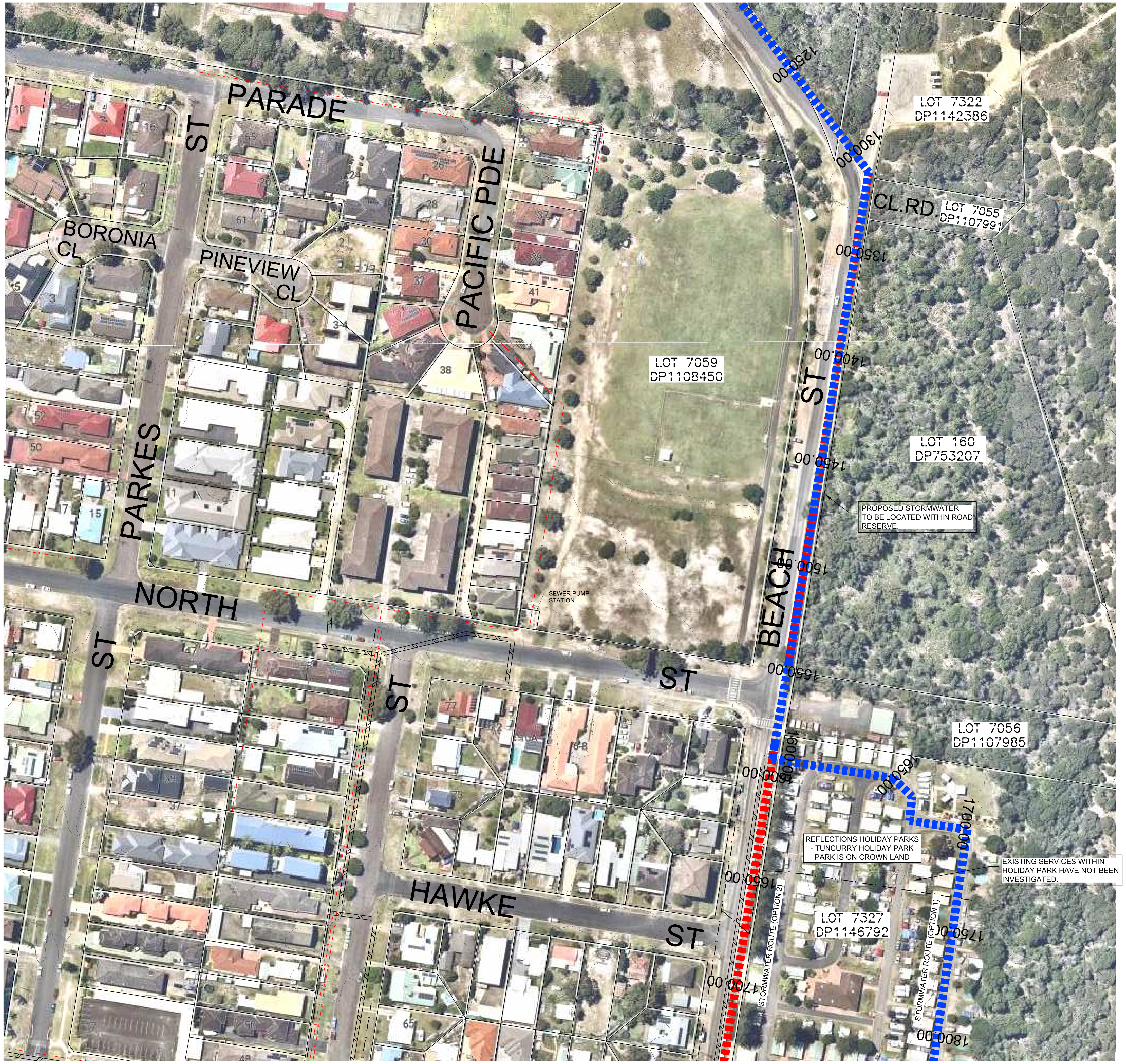
COUNCIL: MID-COAST
DA NUMBER: XXX
DATE: 18/02/21

SHEET 2 OF 8

FILE No.: 3105LD (COMP.JOB 3105DP)

ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE

REFER TO SHEET 2 FOR CONTINUATION



REFER TO SHEET 4 FOR CONTINUATION

MGA

LEGEND

- DENOTES STORMWATER ROUTE (OPTION 1)
- DENOTES STORMWATER ROUTE (OPTION 2)

LEGEND

CODE	DESCRIPTION	LINETYPE
ELECT	UNDERGROUND ELECTRICITY	UG
FCE	FENCE	
HYD /SV	HYDRANT / STOPVALVE	
PIT	EXISTING STORMWATER	
PP /ELP	OVERHEAD ELECTRICITY	O/H
RISNG	RISING MAIN	RM
SMH	SEWER / MANHOLE	S
STN /SSM	SURVEY CONTROL	
SW /PIT	DESIGN STORMWATER	
TEL	TELSTRA	T
TRL	TREELINE	
TR	TREE	
WATER	WATER SUPPLY	W

NOTES

1. RELATIONSHIP OF IMPROVEMENTS TO BOUNDARIES IS DIAGRAMMATIC ONLY. WHERE OFFSETS ARE CRITICAL THEY SHOULD BE CONFIRMED BY FURTHER SURVEY.
2. CONTOURS SHOWN DEPICT THE TOPOGRAPHY EXCEPT AT SPOT LEVELS SHOWN. CONTOURS DO NOT REPRESENT THE EXACT LEVEL AT ANY PARTICULAR POINT.
3. SERVICES SHOWN THEREON HAVE BEEN DETERMINED FROM VISUAL EVIDENCE ONLY. PRIOR TO ANY DEMOLITION, EXCAVATION OR CONSTRUCTION ON THE SITE THE RELEVANT AUTHORITY SHOULD BE CONTACTED TO ESTABLISH DETAILED LOCATION AND DEPTH.
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0 5 10 20 30 40 50
SCALE 1:1000 AT A1

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PRELIMINARY
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HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913
PLOT FILE/DWG No.:
F:\D3105D\3105LD NORTH TUNCURRY.DWG

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687
DRAWN/CHECKED:
MG

PROJECT
NORTH TUNCURRY URBAN RELEASE AREA GRAVITY DRAIN: CONCEPTUAL ALIGNMENT OPTIONS

LSW
SURVEYORS, PLANNERS & ENGINEERS
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PO BOX 510 FORSTER NSW 2428
PH: (02) 6554 7988
EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

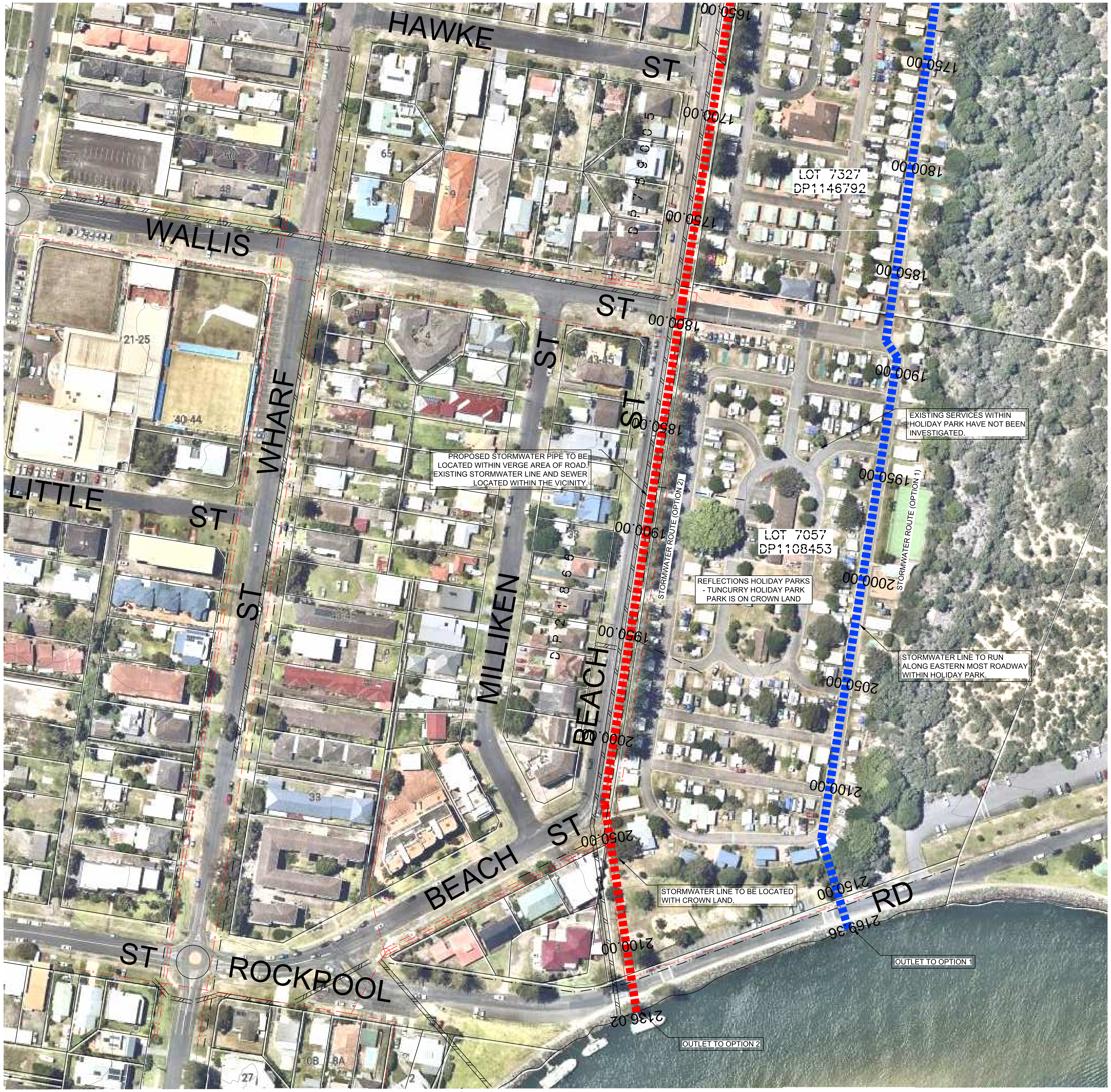
SCALES
HORIZONTAL: 1:1000
VERTICAL: N/A
SHEET SIZE: A1
FIELD SHEETS
Date of survey:

COUNCIL: MID-COAST
DA NUMBER: XXX
DATE: 18/02/21

SHEET 3 OF 8
FILE No.: 3105LD (COMP.JOB 3105DP)

ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE

REFER TO SHEET 3 FOR CONTINUATION



MGA

LEGEND

- DENOTES STORMWATER ROUTE (OPTION 1)
- DENOTES STORMWATER ROUTE (OPTION 2)

LEGEND

CODE	DESCRIPTION	LINETYPE
ELECT	UNDERGROUND ELECTRICITY	UG
FCE	FENCE	
HYD /SV	HYDRANT / STOPVALVE	
PIT	EXISTING STORMWATER	
PP /ELP	OVERHEAD ELECTRICITY	O/H
RISNG	RISING MAIN	RM
SMH	SEWER / MANHOLE	S
STN /SSM	SURVEY CONTROL	
SW /PIT	DESIGN STORMWATER	
TEL	TELSTRA	T
TRL	TREELINE	
TR	TREE	
WATER	WATER SUPPLY	W

NOTES

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0 5 10 20 30 40 50
SCALE 1:1000 AT A1

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HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913

PLOT FILE/DWG No.: F:\D3105D\3105LD NORTH TUNCURRY.DWG

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687

DRAWN/CHECKED: MG

PROJECT
NORTH TUNCURRY URBAN RELEASE AREA GRAVITY DRAIN: CONCEPTUAL ALIGNMENT OPTIONS

LSW SURVEYORS, PLANNERS & ENGINEERS
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PH: (02) 6554 7988
EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

SCALES
HORIZONTAL: 1:1000
VERTICAL: N/A
SHEET SIZE: A1

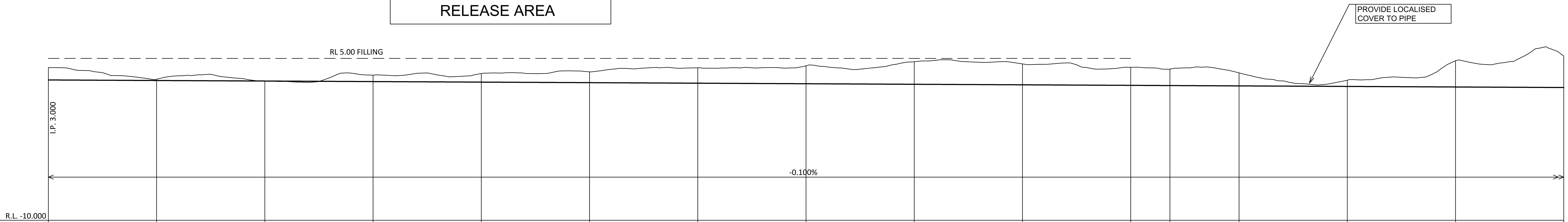
FIELD SHEETS
Date of survey:

COUNCIL: MID-COAST
DA NUMBER: XXX
DATE: 18/02/21

SHEET 4 OF 8
FILE No.: 3105LD (COMP.JOB 3105DP)

ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE

NORTH TUNCURRY URBAN RELEASE AREA



CUT/FILL	-1.144	-0.113	+0.023	-0.586	-0.796	-1.000	-1.422	-1.662	-2.090	-1.937	-1.678	-1.524	-1.207	-0.583	-2.423	-2.917
DESIGN INVERT	3.000	2.950	2.900	2.850	2.800	2.750	2.700	2.650	2.600	2.550	2.500	2.482	2.450	2.400	2.350	2.300
EXISTING SURFACE	4.14	3.06	2.88	3.44	3.60	3.75	4.12	4.31	4.69	4.49	4.18	4.01	3.66	2.98	4.77	5.22
CHAINAGE	0.000	50.000	100.000	150.000	200.000	250.000	300.000	350.000	400.000	450.000	500.000	518.115	550.000	600.000	650.000	700.000

LONGITUDINAL SECTION
GR-STORMWATER OPTION 1C Ch 0.000 to Ch 700.000
SCALES: HORIZONTAL 1:1000 VERTICAL 1:200

CUT/FILL	-2.917	-3.401	-3.551	-3.196	-3.274	-2.774	-1.845	-1.944	-2.926	-3.347	-2.698	-2.647	-2.730
DESIGN INVERT	2.300	2.276	2.250	2.200	2.150	2.100	2.055	2.050	2.036	2.018	2.002	2.000	1.979
EXISTING SURFACE	5.22	5.68	5.80	5.40	5.42	4.87	3.90	3.99	4.96	5.36	4.70	4.65	4.71
CHAINAGE	700.000	723.882	750.000	800.000	850.000	900.000	944.671	950.000	963.723	981.895	998.470	1000.000	1020.911

CUT/FILL	-3.028	-3.017	-3.185	-3.142	-3.296	-3.114	-3.120	-3.126	-3.086	-3.780	-3.852	-3.409	-3.097
DESIGN INVERT	1.950	1.943	1.919	1.900	1.872	1.862	1.850	1.800	1.750	1.700	1.688	1.650	1.600
EXISTING SURFACE	4.98	4.96	5.10	5.04	5.17	4.98	4.97	4.93	4.84	5.48	5.54	5.06	4.70
CHAINAGE	1050.000	1057.205	1081.190	1100.000	1128.171	1137.936	1150.000	1200.000	1250.000	1300.000	1311.656	1350.000	1400.000

LONGITUDINAL SECTION
GR-STORMWATER OPTION 1C Ch 700.000 to Ch 1400.000
SCALES: HORIZONTAL 1:1000 VERTICAL 1:200

OPTION 1 GRAVITY PIPE ALIGNMENT

Constraints	Comment
CH 1600 pipe enters Caravan Park	
CH 1700-1750	Existing Telstra / NBN crossing
CH 2150 - Outlet	New pipe crossing existing sewer & Telstra (Rockpool Road)

OPTION 1 GRAVITY PIPE ALIGNMENT - LANDOWNERSHIP DETAILS

Constraints	Comment
CH 0 - CH 1000	Crown Land
CH 1000 - CH 1250	Public Road Reserve (Beach Street)
CH 1250 - CH 1300	Crown Land
CH 1300 - CH 1600	Public Road Reserve (Beach Street)
CH 1600 - CH 2150	Reflections Holiday Parks - Tuncurry Holiday Park (Crown land)
CH 2150 - CH 2169.36 (Outlet)	Public Road Reserve (Rockpool Road)

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NOTE: EXISTING SURFACE GENERATED UTILISING LIDAR. SUBJECT TO FUTURE DETAILED SURVEY

PRELIMINARY
SUBJECT TO APPROVAL, FINAL SURVEY AND DESIGN

HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913
PLOT FILE/DWG No.:
F:\D3105D\3105LD NORTH TUNCURRY.DWG

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687XXX
DRAWN/CHECKED:
MG

PROJECT
NORTH TUNCURRY URBAN RELEASE AREA
GRAVITY DRAIN:
CONCEPTUAL ALIGNMENT OPTIONS
STORMWATER ROUTE (OPTION 1) LONGSECTION

LSW SURVEYORS, PLANNERS & ENGINEERS
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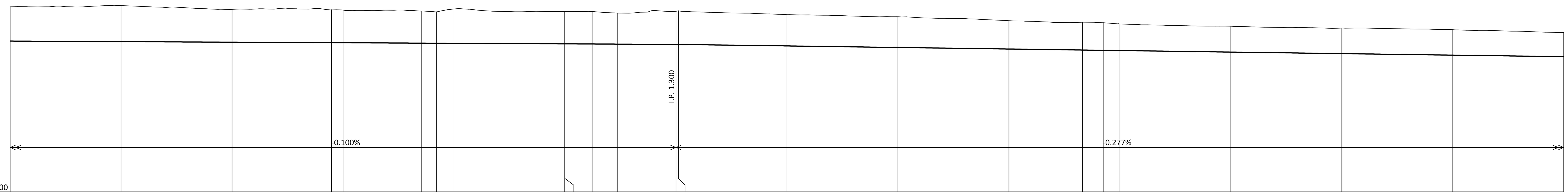
EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

SCALES
HORIZONTAL: 1:1000
VERTICAL: 1:200
SHEET SIZE: A1
FIELD SHEETS
Date of survey:

COUNCIL: MID-COAST
DA NUMBER: XXX
DATE: 18/02/21

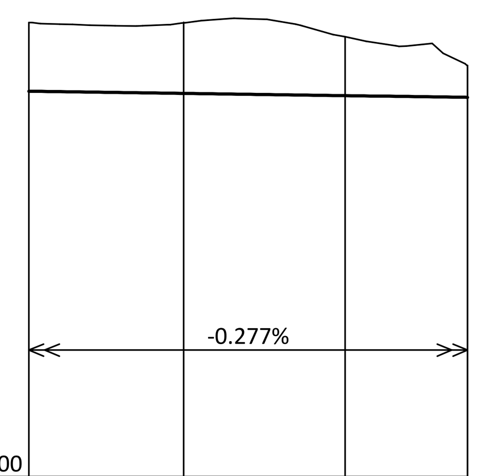
SHEET 5 OF 8
FILE No.: 3105LD (COMP.JOB 3105DP)

ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE



CHAINAGE	CUT/FILL	DESIGN INVERT	EXISTING SURFACE
1400.000	-3.097	1.600	4.70
1450.000	-3.256	1.550	4.81
1500.000	-2.963	1.500	4.46
1544.870	-2.960	1.455	4.42
1550.000	-2.948	1.450	4.40
1585.237	-2.891	1.415	4.31
1592.053	-2.824	1.408	4.23
1600.000	-3.098	1.400	4.50
1649.886	-2.917	1.350	4.27
1650.000	-2.917	1.350	4.27
1662.241	-2.929	1.338	4.27
1673.545	-2.801	1.326	4.13
1700.000	-2.991	1.300	4.29
1701.089	-3.027	1.297	4.32
1750.000	-2.824	1.162	3.99
1800.000	-2.769	1.023	3.79
1850.000	-2.557	0.885	3.44
1883.112	-2.509	0.793	3.30
1892.747	-2.484	0.766	3.25
1900.000	-2.398	0.746	3.14
1950.000	-2.333	0.608	2.94
2000.000	-2.304	0.469	2.77
2050.000	-2.294	0.331	2.63
2100.000	-2.178	0.192	2.37

LONGITUDINAL SECTION
GR-STORMWATER OPTION 1C Ch 1400.000 to Ch 2100.000
SCALES: HORIZONTAL 1:1000 VERTICAL 1:200



CHAINAGE	CUT/FILL	DESIGN INVERT	EXISTING SURFACE
2100.000	-2.178	0.192	2.37
2124.483	-2.230	0.124	2.35
2150.000	-1.862	0.054	1.92
2169.355	-0.992	0.000	0.99

LONGITUDINAL SECTION
GR-STORMWATER OPTION 1C Ch 2100.000 to Ch 2169.355
SCALES: HORIZONTAL 1:1000 VERTICAL 1:200

OPTION 1 GRAVITY PIPE ALIGNMENT

Constraints	Comment
CH 1600 pipe enters Caravan Park	
CH 1700-1750	Existing Telstra / NBN crossing
CH 2150 - Outlet	New pipe crossing existing sewer & Telstra (Rockpool Road)

OPTION 1 GRAVITY PIPE ALIGNMENT - LANDOWNERSHIP DETAILS

Constraints	Comment
CH 0 - CH 1000	Crown Land
CH 1000 - CH 1250	Public Road Reserve (Beach Street)
CH 1250 - CH 1300	Crown Land
CH 1300 - CH 1600	Public Road Reserve (Beach Street)
CH 1600 - CH 2150	Reflections Holiday Parks - Tuncurry Holiday Park (Crown Land)
CH 2150 - CH 2169.36 (Outlet)	Public Road Reserve (Rockpool Road)

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NOTE: EXISTING SURFACE GENERATED UTILISING LIDAR. SUBJECT TO FUTURE DETAILED SURVEY

PRELIMINARY
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HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913

PLOT FILE/DWG No.:
F:\D3105D\3105LD NORTH TUNCURRY.DWG

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687XXX

DRAWN/CHECKED:
MG

PROJECT
NORTH TUNCURRY URBAN RELEASE AREA GRAVITY DRAIN: CONCEPTUAL ALIGNMENT OPTIONS STORMWATER ROUTE (OPTION 1) LONGSECTION

LSW SURVEYORS, PLANNERS & ENGINEERS
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PO BOX 510 FORSTER NSW 2428
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EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

SCALES
HORIZONTAL: 1:1000
VERTICAL: 1:200
SHEET SIZE: A1

FIELD SHEETS
Date of survey:

COUNCIL: MID-COAST
DA NUMBER: XXX

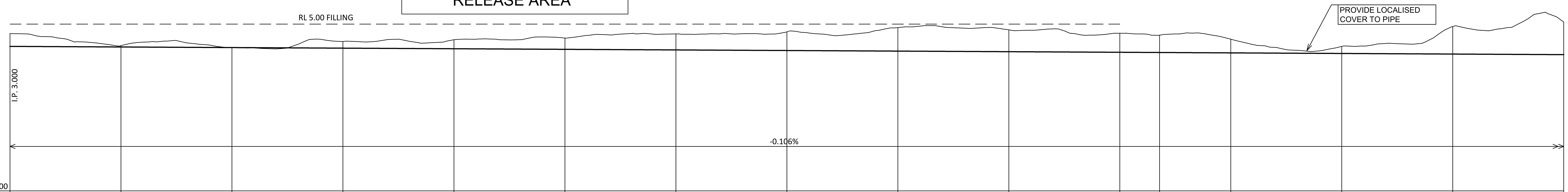
DATE: 18/02/21

SHEET 6 OF 8

FILE No.: 3105LD (COMP.JOB 3105DP)

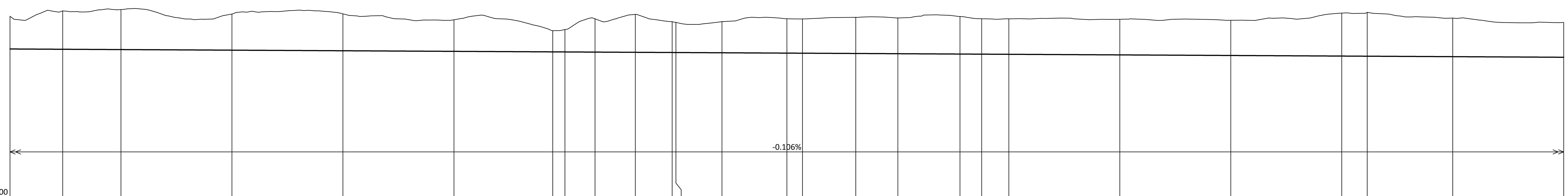
ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE

NORTH TUNCURRY URBAN RELEASE AREA



CUT/FILL	-1.144	-0.116	+0.017	-0.595	-0.809	-1.016	-1.442	-1.684	-2.116	-1.964	-1.709	-1.551	-1.236	-0.633	-2.473	-2.936
DESIGN INVERT	3.000	2.947	2.894	2.841	2.788	2.734	2.681	2.628	2.575	2.522	2.469	2.450	2.416	2.363	2.309	2.256
EXISTING SURFACE	4.14	3.06	2.88	3.44	3.60	3.75	4.12	4.31	4.69	4.49	4.18	4.00	3.65	3.00	4.78	5.19
CHAINAGE	0.000	50.000	100.000	150.000	200.000	250.000	300.000	350.000	400.000	450.000	500.000	517.929	550.000	600.000	650.000	700.000

LONGITUDINAL SECTION
GR-STORMWATER OPTION 2B Ch 0.000 to Ch 700.000
SCALES: HORIZONTAL 1:1000 VERTICAL 1:200



CUT/FILL	-2.936	-3.446	-3.600	-3.252	-3.319	-2.836	-1.904	-2.009	-2.986	-3.408	-2.761	-2.702	-2.794	-3.093
DESIGN INVERT	2.256	2.231	2.203	2.150	2.097	2.044	1.996	1.991	1.976	1.957	1.939	1.938	1.915	1.884
EXISTING SURFACE	5.19	5.68	5.80	5.40	5.42	4.88	3.90	4.00	4.96	5.36	4.70	4.64	4.71	4.98
CHAINAGE	700.000	723.719	750.000	800.000	850.000	900.000	944.509	950.000	963.560	981.733	998.308	1000.000	1020.748	1050.000

CUT/FILL	-3.083	-3.253	-3.213	-3.367	-3.185	-3.192	-3.202	-3.165	-3.863	-3.933	-3.464	-3.123
DESIGN INVERT	1.877	1.851	1.831	1.801	1.791	1.778	1.725	1.672	1.619	1.607	1.566	1.513
EXISTING SURFACE	4.96	5.10	5.04	5.17	4.98	4.97	4.93	4.84	5.48	5.54	5.03	4.63
CHAINAGE	1057.043	1081.028	1100.000	1128.009	1137.773	1150.000	1200.000	1250.000	1300.000	1311.494	1350.000	1400.000

LONGITUDINAL SECTION
GR-STORMWATER OPTION 2B Ch 700.000 to Ch 1400.000
SCALES: HORIZONTAL 1:1000 VERTICAL 1:200

OPTION 2 GRAVITY PIPE ALIGNMENT	
Constraints	Comment
APPROX. CH 1600	Existing stormwater 2 x 750 dia stormwater pipes
	run parallel to new pipe. Telstra / NBN running adjacent
	to boundary
CH 1650-1700	Telstra / NBN crossing Beach Street
CH 1750-1800 (Wallis Street Intersection)	Telstra / NBN & sewer crossing new pipe alignment.
	Southern side of intersection additional sewer line crossing.
CH 1800-2050	2 x Sewer, Stormwater & Telstra / NBN run parallel to new pipe
CH 2050	Crosses existing sewer
CH 2100 - Outlet	New pipe crossing existing sewer & Telstra (Rockpool Road)

OPTION 2 GRAVITY PIPE ALIGNMENT - LANDOWNERSHIP DETAILS	
Constraints	Comment
CH 0 - CH 1000	Crown Land
CH 1000 - CH 1250	Public Road Reserve (Beach Street)
CH 1250 - CH 1300	Crown Land
CH 1300 - CH 2050	Public Road Reserve (Beach Street)
CH 2050 - CH 2100	Reflections Holiday Parks - Tuncurry Holiday Park (Crown Land)
CH 2100 - CH 2136.02 (Outlet)	Public Road Reserve (Rockpool Road)

NOTE: EXISTING SURFACE GENERATED UTILISING LIDAR. SUBJECT TO FUTURE DETAILED SURVEY

PRELIMINARY
SUBJECT TO APPROVAL, FINAL SURVEY AND DESIGN

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NOTE: The contractor shall check and verify all works on site (including works by others) before commencing any works. All discrepancies are to be reported to the Project Manager or Lidbury, Summers & Whiteman prior to commencing work. Do not scale dimensions from the plan. All dimensions to be confirmed by dimensions from the plan. All dimensions to be confirmed by Lidbury, Summers & Whiteman.

HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687

PLOT FILE/DWG No.:
F:\D3105D\3105LD NORTH TUNCURRY.DWG

DRAWN/CHECKED:
MG

PROJECT
NORTH TUNCURRY URBAN RELEASE AREA
GRAVITY DRAIN:
CONCEPTUAL ALIGNMENT OPTIONS
STORMWATER ROUTE (OPTION 2) LONGSECTION

LSW
SURVEYORS, PLANNERS & ENGINEERS
LIDBURY, SUMMERS & WHITEMAN
1st FLOOR, 3 WHARF ST. FORSTER 2428
PO BOX 510 FORSTER NSW 2428
PH: (02) 6554 7988
EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

SCALES
HORIZONTAL: 1:1000
VERTICAL: 1:200
SHEET SIZE: A1

FIELD SHEETS
Date of survey:

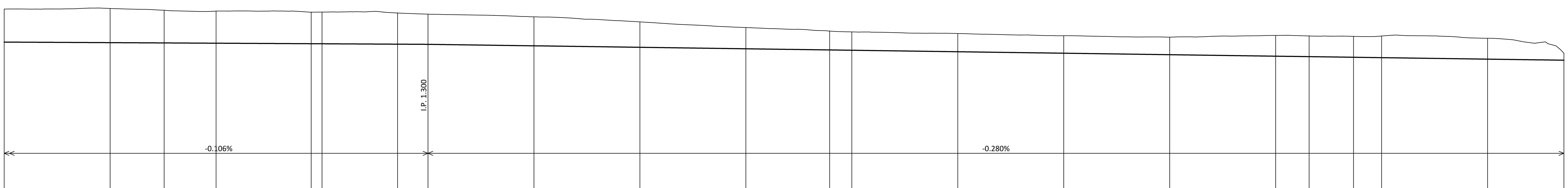
COUNCIL: MID-COAST
DA NUMBER: XXX
DATE: 18/02/21

FILE No.: 3105LD (COMP.JOB 3105DP)

ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE

OPTION 2 GRAVITY PIPE ALIGNMENT - LANDOWNERSHIP DETAILS	
Constraints	Comment
CH 0 - CH 1000	Crown Land
CH 1000 - CH 1250	Public Road Reserve (Beach Street)
CH 1250 - CH 1300	Crown Land
CH 1300 - CH 2050	Public Road Reserve (Beach Street)
CH 2050 - CH 2100	Reflections Holiday Parks - Tuncurry Holiday Park (Crown Land)
CH 2100 - CH 2136.02 (Outlet)	Public Road Reserve (Rockpool Road)

OPTION 2 GRAVITY PIPE ALIGNMENT	
Constraints	Comment
APPROX. CH 1600	Existing stormwater 2 x 750 dia stormwater pipes
	run parallel to new pipe. Telstra / NBN running adjacent
	to boundary
CH 1650-1700	Telstra / NBN crossing Beach Street
CH 1750-1800 (Wallis Street Intersection)	Telstra / NBN & sewer crossing new pipe alignment.
	Southern side of intersection additional sewer line crossing.
CH 1800-2050	2 x Sewer, Stormwater & Telstra / NBN run parallel to new pipe
CH 2050	Crosses existing sewer
CH 2100 - Outlet	New pipe crossing existing sewer & Telstra (Rockpool Road)



R.L. -13.000

CHAINAGE	1400.000	1450.000	1475.495	1500.000	1544.883	1550.000	1585.637	1600.000	1650.000	1700.000	1750.000
CUT/FILL	-3.123	-3.229	-3.087	-3.031	-2.977	-2.991	-2.947	-2.852	-2.740	-2.402	-2.008
DESIGN INVERT	1.513	1.459	1.432	1.406	1.359	1.353	1.315	1.300	1.160	1.020	0.880
EXISTING SURFACE	4.63	4.69	4.52	4.44	4.34	4.34	4.26	4.15	3.90	3.42	2.89

CHAINAGE	1789.550	1800.000	1850.000	1900.000	1950.000	2000.000	2015.807	2036.738	2050.000	2100.000	2136.019
CUT/FILL	-1.783	-1.734	-1.725	-1.651	-1.657	-1.960	-1.954	-1.969	-2.045	-1.974	-0.658
DESIGN INVERT	0.770	0.740	0.600	0.460	0.321	0.181	0.136	0.078	0.041	-0.099	-0.200
EXISTING SURFACE	2.55	2.47	2.32	2.11	1.98	2.14	2.09	2.05	2.09	1.87	0.46

LONGITUDINAL SECTION
GR-STORMWATER OPTION 2B Ch 1400.000 to Ch 2136.019
SCALES: HORIZONTAL 1:1000 VERTICAL 1:200

NOTE: EXISTING SURFACE GENERATED UTILISING LIDAR. SUBJECT TO FUTURE DETAILED SURVEY

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HORIZONTAL
CO-ORDINATE SYSTEM: MGA LOCAL
MARKS ADOPTED: SSM 136468
EAST: 452162.737 NORTH: 6442031.913

PLOT FILE/DWG No.:
F:\D3105D\3105LD NORTH TUNCURRY.DWG

VERTICAL
DATUM: AHD
BENCHMARK: SSM136468
R.L.: 5.687

DRAWN/CHECKED:
MG

PROJECT
NORTH TUNCURRY URBAN RELEASE AREA
GRAVITY DRAIN:
CONCEPTUAL ALIGNMENT OPTIONS
STORMWATER ROUTE (OPTION 2) LONGSECTION

LSW
SURVEYORS, PLANNERS & ENGINEERS
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PO BOX 510 FORSTER NSW 2428
PH: (02) 6554 7988
EMAIL: consult@lswsurveyors.com.au WEBSITE: www.lswsurveyors.com.au

SCALES
HORIZONTAL: 1:1000
VERTICAL: 1:200
SHEET SIZE: A1

FIELD SHEETS
Date of survey:

COUNCIL: MID-COAST

DA NUMBER:
XXX

DATE:
18/02/21

SHEET 8 OF 8

FILE No.:
3105LD
(COMP.JOB 3105DP)

ISSUE	DATE	COMMENTS
A	19/02/21	FIRST ISSUE
B	16/03/21	SECOND ISSUE
C	22/03/21	THIRD ISSUE

Appendix B

PMP calculation sheets

GSDM WORKSHEET

LOCATION INFORMATION				
Catchment Name: NTURA			State: NSW	
Duration Limit:	6	(3-6 hours)	Area (km^2):	4
Approx. Centroid:	Latitude °S:		Longitude °E:	
Portion of Area Considered:				
Smooth(0.0 - 1.0), S =	0		**Rough (0.0 -1.0), R =**	1
ELEVATION ADJUSTMENT FACTOR (EAF)				
Mean Elevation (m):	5	required if greater than 1,500 m		
Adjustment for Elevation:	0.00	-0.05 per 300 m above 1500 m		
EAF (0.85 - 1.00)=	1.00			
GSDM MOISTURE ADJUSTMENT FACTOR (MAF)				
$EPW_{catchment}$=	0.75	GSDM MAF=EPWcatchment/104.5		
OR read directly off GSDM Moisture Adjustment Factor chart at centroid				
GSDM MAF (0.46-1.19)=	0.75			

PMP VALUES (mm)

Duration (hours)	Initial Depth - Smooth(DS)	Initial Depth - Rough(DR)	PMP Estimate = (DSHS + DRHR)H MAF H EAF	Rounded PMP Estimate(nearest 10 mm)
0.25	226	226	169	170
0.5	329	329	247	250
0.75	417	417	313	310
1	485	485	364	360
1.5	553	624	468	470
2	618	730	548	550
2.5	658	805	604	600
3	693	882	662	660
4	759	1011	758	760
5	818	1112	834	830
6	866	1178	884	880

Prepared by Chris Kuczera	Date: 14/12/2020
Checked by Jason O'Brien	Date: 17/02/2021

WORKSHEET 2: Generalised Southeast Australia Storm Method (GSAM)

LOCATION INFORMATION			
Catchment Name:	NTURA	State:	NSW
GSAM zone:	GSAM - GTSMR Coastal Transition Zone	Area (km^2):	4

CATCHMENT FACTORS			
Topographical Adjustment Factor	**TAF =**	1.08	(1.0 - 2.0)

Annual Moisture Adjustment Factor

$$MAF = \frac{EPW_{\text{seasonal catchment average}}}{EPW_{\text{seasonal standard}}}$$

Season	EPWseasonal catchment average	EPWseasonal standard	MAF	
Summer (annual)	78	80.8	0.97	(0.6 - 1.05)
Autumn	64	71	0.90	(0.56 - 0.91)

Summer PMP values (mm)			Autumn PMP values (mm)		
Duration (hours)	Initial Depth (D_{summer})	PMP Estimate (D_sxTAFxMAF$_s$)	Duration (hours)	Initial Depth (D_{autumn})	PMP Estimate (D_axTAFxMAF$_a$)
24	904	943	24	593	577
36	1012	1055	36	725	706
48	1065	1111	48	851	829
72	1114	1161	72	1073	1045
96	1153	1202	96	1151	1121

Final GSAM PMP Estimates			
Duration (hours)	Maximum of the Seasonal Depths	Preliminary PMP Estimate (nearest 10 mm)	Final PMP Estimate
1	360	360	360
2	550	550	550
3	660	660	660
4	760	760	760
5	830	830	830
6	880	880	880
12	(no preliminary estimates available)		910
24	943	940	940
36	1055	1050	1050
48	1111	1110	1110
72	1161	1160	1160
96	1202	1200	1200

Prepared by Chris Kuczera	Date: 14/12/2020
Checked by Jason O'Brien	Date: 17/02/2021

WORKSHEET 2: Generalised Tropical Storm Method Revised (GTSMR)

LOCATION INFORMATION

Catchment Name:	NTURA	State:	NSW
GTSMR zone:	GSAM - GTSMR Coastal Transition Zone	Area (km^2):	4

CATCHMENT FACTORS

Topographical Adjustment Factor	**TAF =**	1.08	**(1.0 - 2.0)**
Decay Amplitude Factor	**DAF =**	0.84	**(0.7 - 1.0)**

Annual Moisture Adjustment Factor

$MAF_a = EPW_{catchment}/120.00$

$MAF_w = EPW_{catchment\_winter}/82.30$

Season	**EPW catchment**	**EPW standard**	**MAF**	
Annual	78	120	**0.65**	**(0.4 - 1.1)**
Winter	54	82.3	**0.66**	**(0.4 - 1.1)**

Summer (Annual) PMP values (mm)			**Winter PMP values (mm)**		
Duration (hours)	Initial Depth (D_w)	PMP Estimate $=D_w x TAF x DAF x MAF_w$	Duration (hours)	Initial Depth (D_{autumn})	PMP Estimate ($D_a x TAF x MAF_a$)
24	**1371**	**808**	24	**840**	**500**
36	**1683**	**992**	36	**1008**	**600**
48	**1972**	**1163**	48	**1109**	**660**
72	**2484**	**1465**	72	**1239**	**738**
96	**2780**	**1639**	96	**1281**	**762**
120	**2924**	**1724**	120	**1322**	**787**

Final GSAM PMP Estimates

Duration (hours)	Maximum of the Seasonal Depths	Preliminary PMP Estimate (nearest 10 mm)	Final PMP Estimate
1	**360**	360	360
2	**550**	550	550
3	**660**	660	660
4	**760**	760	760
5	**830**	830	830
6	**880**	880	880
12	(no preliminary estimates available)		845
24	**808**	**810**	810
36	**992**	**990**	990
48	**1163**	**1160**	1160
72	**1465**	**1460**	1460
96	**1639**	**1640**	1640
120	**1724**	**1720**	1720

Prepared by Chris Kuczera	Date: 14/12/2020
Checked by Jason O'Brien	Date: 17/02/2021

Appendix C

Design storm analysis results

C.1 Ensemble storm results

C.1.1 Design storm scenarios

Design storm results

3.5065 denotes the adopted ensemble

1% AEP design storms

	Storm duration								
Ensemble	**12 hrs**	**24 hrs**	**36 hr**	**48 hr**	**72hr**	**96hr**	**120 hr**	**144 hr**	**168 hr**
1	3.51	3.69	3.77	3.78	3.87	3.97	4.08	4.19	4.18
2	3.51	3.70	3.77	3.92	3.83	3.89	4.07	4.02	4.03
3	3.50	3.69	3.78	3.83	3.86	3.89	4.01	4.21	4.22
4	3.51	3.67	3.84	3.88	3.80	3.85	3.87	3.91	3.94
5	3.51	3.70	3.83	3.85	3.94	3.92	4.01	4.01	3.86
6	3.51	3.68	3.76	3.84	3.93	4.05	4.02	4.04	4.02
7	3.51	3.68	3.78	3.86	3.83	3.95	4.01	4.05	3.97
8	3.51	3.69	3.79	3.86	3.87	4.02	3.85	3.85	4.00
9	3.51	3.69	3.84	3.84	3.91	3.84	3.95	3.80	4.05
10	3.51	3.70	3.81	3.81	3.97	3.89	3.82	4.16	4.10
Median	**3.51**	**3.69**	**3.79**	**3.85**	**3.87**	**3.91**	**4.01**	**4.03**	**4.02**

0.2% AEP design storms

	Storm duration								
Ensemble	**12 hrs**	**24 hrs**	**36 hr**	**48 hr**	**72hr**	**96hr**	**120 hr**	**144 hr**	**168 hr**
1	3.72	3.94	4.04	4.11	4.25	4.34	4.45	4.55	4.53
2	3.72	3.96	4.04	4.23	4.23	4.28	4.42	4.39	4.41
3	3.72	3.94	4.06	4.17	4.24	4.23	4.34	4.54	4.57
4	3.73	3.91	4.10	4.21	4.21	4.25	4.20	4.28	4.32
5	3.74	3.96	4.10	4.19	4.32	4.29	4.37	4.37	4.13
6	3.72	3.92	4.05	4.15	4.29	4.39	4.29	4.42	4.36
7	3.72	3.91	4.05	4.17	4.22	4.31	4.37	4.30	4.34
8	3.73	3.94	4.04	4.19	4.25	4.38	4.21	4.22	4.38
9	3.74	3.93	4.10	4.19	4.29	4.22	4.26	4.12	4.35
10	3.74	3.96	4.09	4.12	4.34	4.29	4.15	4.49	4.37
Median	**3.73**	**3.94**	**4.05**	**4.18**	**4.25**	**4.29**	**4.32**	**4.38**	**4.37**

0.05% AEP design storms

	Storm duration								
Ensemble	**12 hrs**	**24 hrs**	**36 hr**	**48 hr**	**72hr**	**96hr**	**120 hr**	**144 hr**	**168 hr**
1	3.94	4.19	4.34	4.40	4.55	4.64	4.73	4.81	4.80
2	3.93	4.22	4.33	4.51	4.52	4.59	4.70	4.70	4.73
3	3.94	4.20	4.35	4.45	4.53	4.52	4.63	4.80	4.83
4	3.95	4.17	4.39	4.49	4.50	4.56	4.46	4.57	4.63
5	3.96	4.21	4.38	4.47	4.61	4.61	4.68	4.66	4.36
6	3.93	4.18	4.35	4.42	4.59	4.66	4.53	4.73	4.66
7	3.93	4.17	4.35	4.44	4.50	4.62	4.67	4.59	4.66
8	3.94	4.19	4.32	4.47	4.56	4.67	4.51	4.54	4.69
9	3.97	4.18	4.38	4.47	4.59	4.52	4.56	4.46	4.64
10	3.96	4.20	4.38	4.39	4.63	4.60	4.48	4.75	4.67
Median	**3.94**	**4.19**	**4.35**	**4.46**	**4.55**	**4.60**	**4.59**	**4.68**	**4.67**

C.1.2 Sensitivity scenarios

Sensitivity Scenario 1 - 1% design storms only									
				Storm duration					
Ensemble	12 hrs	24 hrs	36 hr	48 hr	72hr	96hr	120 hr	144 hr	168 hr
1				3.87			4.20	4.28	4.29
2				3.95			4.20	4.21	4.22
3				3.91			4.14	4.29	4.30
4				3.93			4.07	4.06	4.10
5				3.92			4.17	4.12	4.08
6				3.91			4.15	4.21	4.15
7				3.92			4.17	4.19	4.13
8				3.92			4.08	4.10	4.17
9				3.90			4.13	4.06	4.21
10				3.89			4.06	4.25	4.24
Median				**3.91**			**4.15**	**4.20**	**4.19**

Sensitivity Scenario 2 - 1% design storms only									
				Storm duration					
Ensemble	12 hrs	24 hrs	36 hr	48 hr	72hr	96hr	120 hr	144 hr	168 hr
1				4.01			4.33	4.40	4.43
2				4.01			4.34	4.38	4.41
3				4.01			4.33	4.40	4.43
4				4.01			4.33	4.39	4.43
5				4.01			4.34	4.36	4.42
6				4.01			4.35	4.38	4.40
7				4.01			4.33	4.40	4.42
8				4.01			4.32	4.37	4.42
9				4.01			4.34	4.37	4.44
10				**4.01**			**4.33**	**4.39**	**4.45**
Median				**4.01**			**4.33**	**4.38**	**4.42**

Sensitivity Scenario 3 - 1% design storms only									
				Storm duration					
Ensemble	12 hrs	24 hrs	36 hr	48 hr	72hr	96hr	120 hr	144 hr	168 hr
1				4.21			4.43	4.46	4.40
2				4.32			4.36	4.31	4.32
3				4.26			4.29	4.45	4.46
4				4.29			4.11	4.33	4.36
5				4.28			4.34	4.34	4.07
6				4.24			4.19	4.36	4.30
7				4.25			4.33	4.20	4.30
8				4.28			4.13	4.13	4.33
9				4.28			4.22	4.02	4.24
10				4.21			4.10	4.40	4.25
Median				**4.27**			**4.26**	**4.33**	**4.31**

Sensitivity Scenario 4 - 1% design storms only									
					Storm duration				
Ensemble	12 hrs	24 hrs	36 hr	48 hr	72hr	96hr	120 hr	144 hr	168 hr
1				4.33			4.58	4.61	4.58
2				4.39			4.54	4.53	4.54
3				4.36			4.49	4.60	4.61
4				4.38			4.41	4.45	4.49
5				4.37			4.54	4.49	4.33
6				4.35			4.43	4.57	4.49
7				4.35			4.53	4.46	4.49
8				4.37			4.42	4.43	4.51
9				4.37			4.46	4.38	4.50
10				4.33			4.41	4.57	4.50
Median				**4.36**			**4.47**	**4.51**	**4.50**

Sensitivity Scenario 5 - 1% design storms only									
					Storm duration				
Ensemble	12 hrs	24 hrs	36 hr	48 hr	72hr	96hr	120 hr	144 hr	168 hr
1				4.46			4.72	4.77	4.79
2				4.49			4.72	4.75	4.77
3				4.48			4.70	4.77	4.79
4				4.48			4.70	4.76	4.79
5				4.48			4.72	4.74	4.74
6				4.47			4.70	4.76	4.76
7				4.47			4.71	4.73	4.77
8				4.48			4.69	4.72	4.78
9				4.48			4.70	4.71	4.77
10				4.47			4.69	4.76	4.79
Median				**4.48**			**4.70**	**4.75**	**4.78**

C.2 Basin water level hydrographs

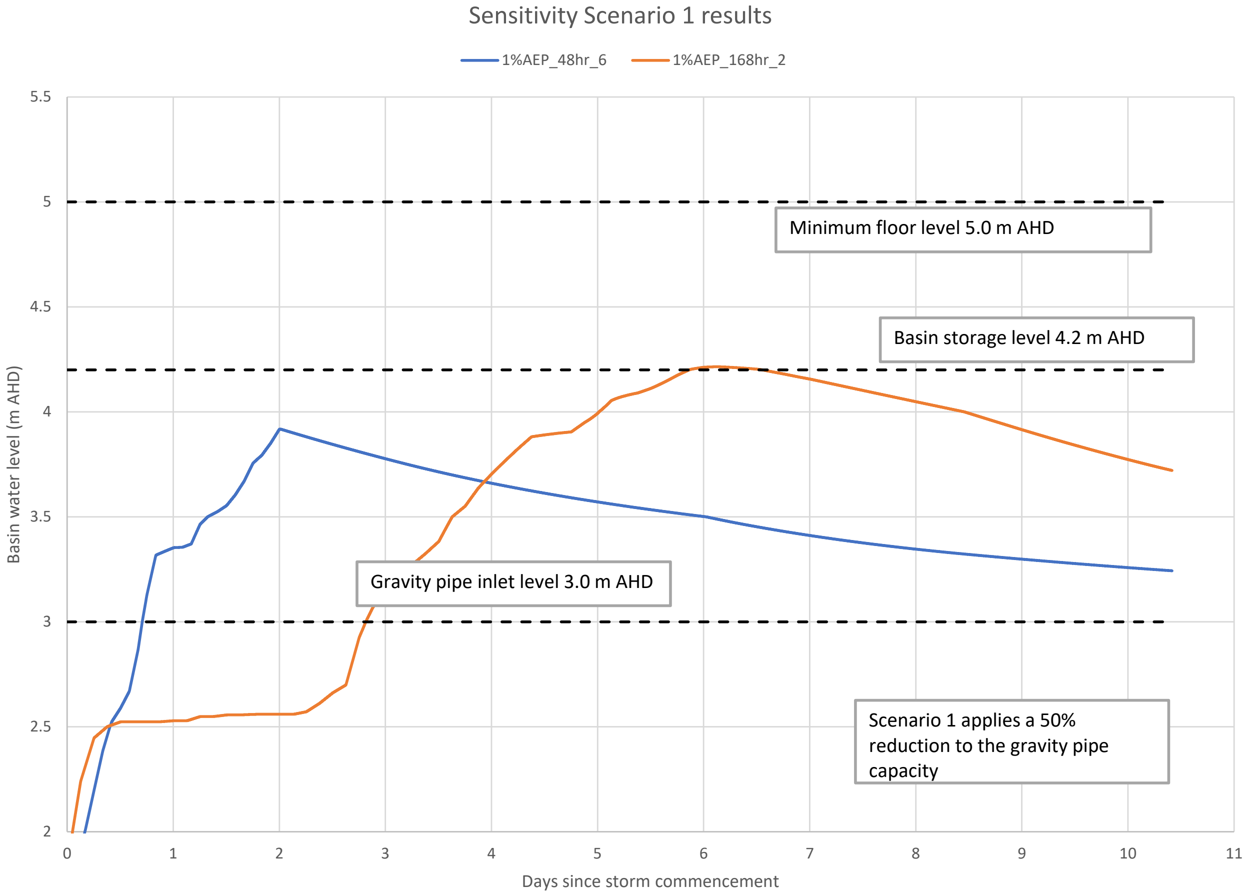

Figure C.1 Basin water level hydrographs – 1% AEP event: Sensitivity Scenario 1

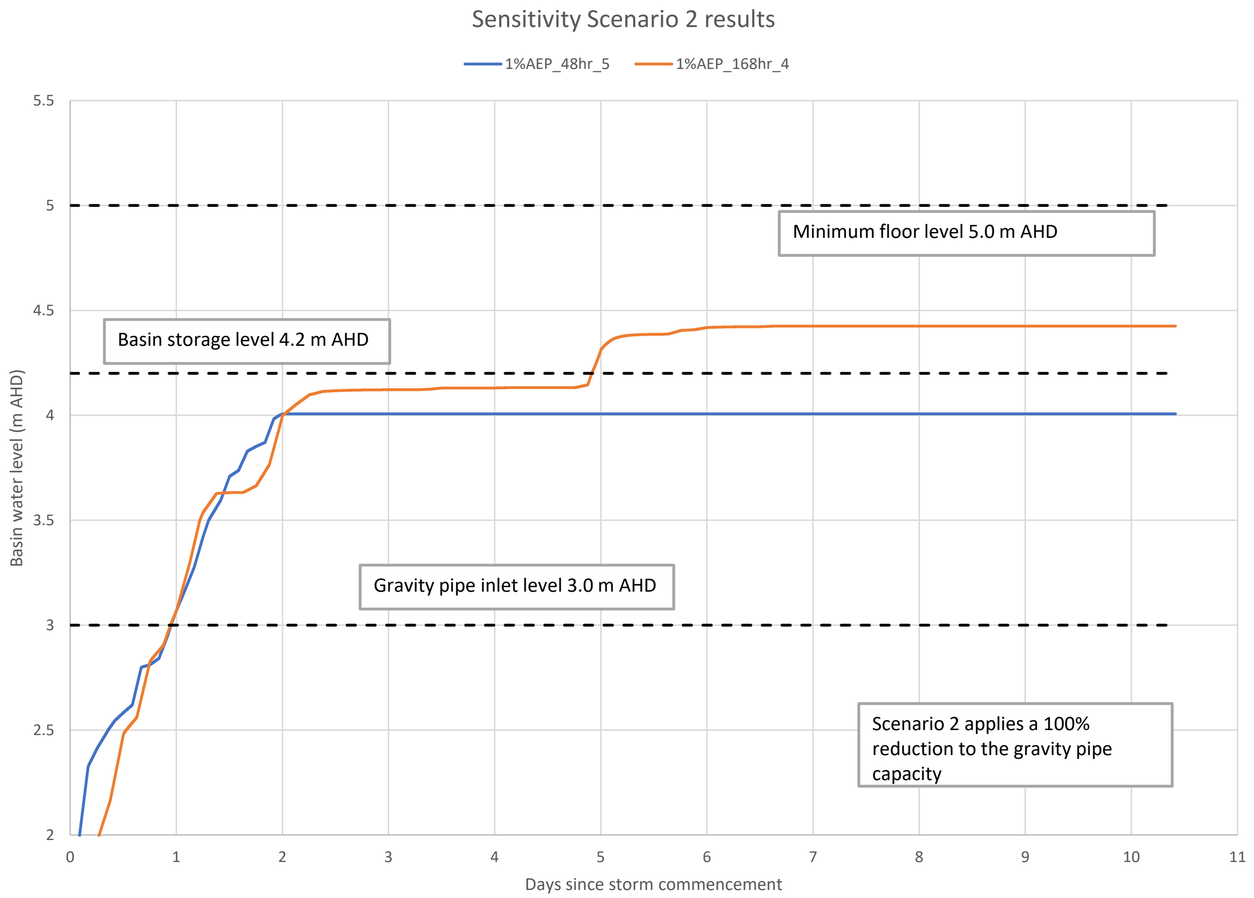
Sensitivity Scenario 2 results
1%AEP_48hr_5
1%AEP_168hr_4
Basin water level (m AHD)
Days since storm commencement
Minimum floor level 5.0 m AHD
Basin storage level 4.2 m AHD
Gravity pipe inlet level 3.0 m AHD
Scenario 2 applies a 100% reduction to the gravity pipe capacity
5.5
5
4.5
4
3.5
3
2.5
2
0
1
2
3
4
5
6
7
8
9
10
11

Figure C.2 Basin water level hydrographs – 1% AEP event: Sensitivity Scenario 2

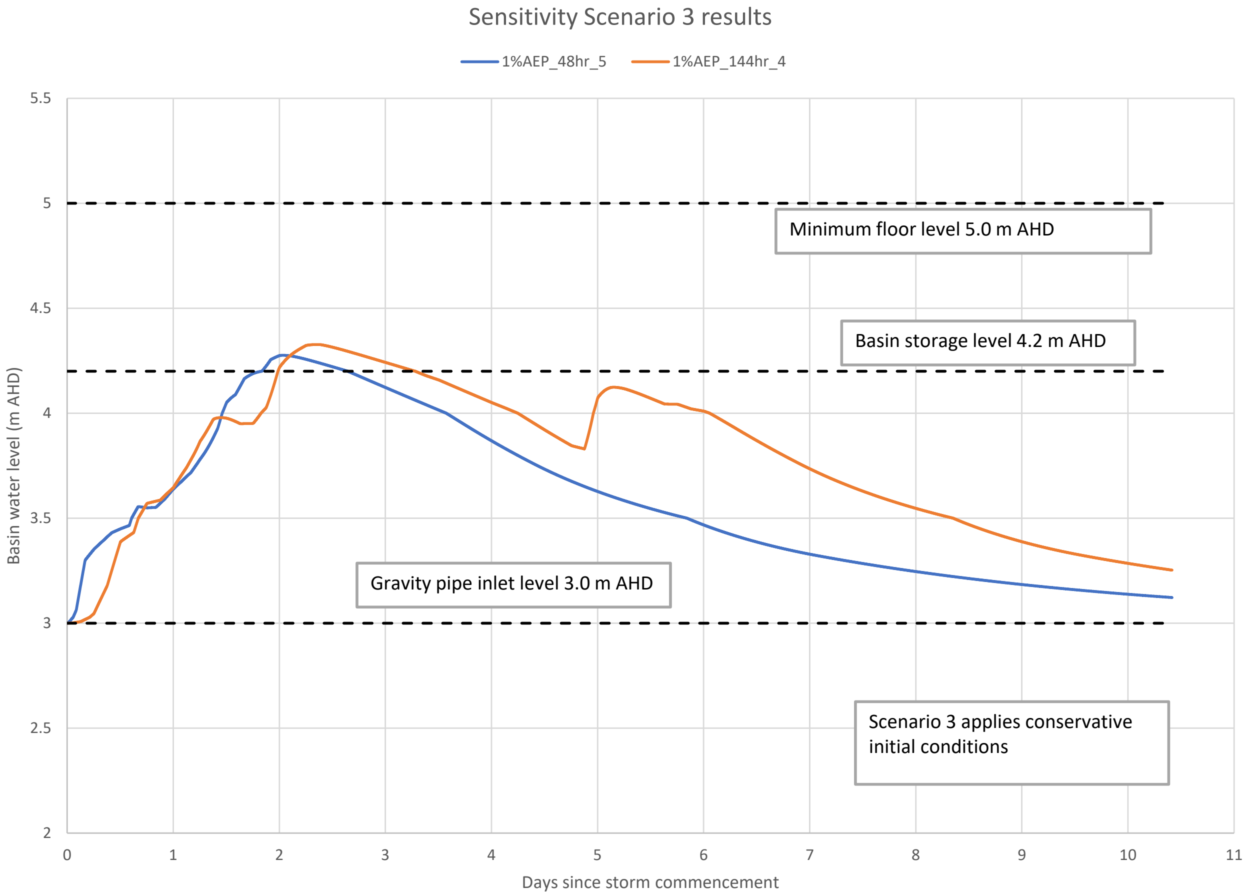

Figure C.3 Basin water level hydrographs – 1% AEP event: Sensitivity Scenario 3

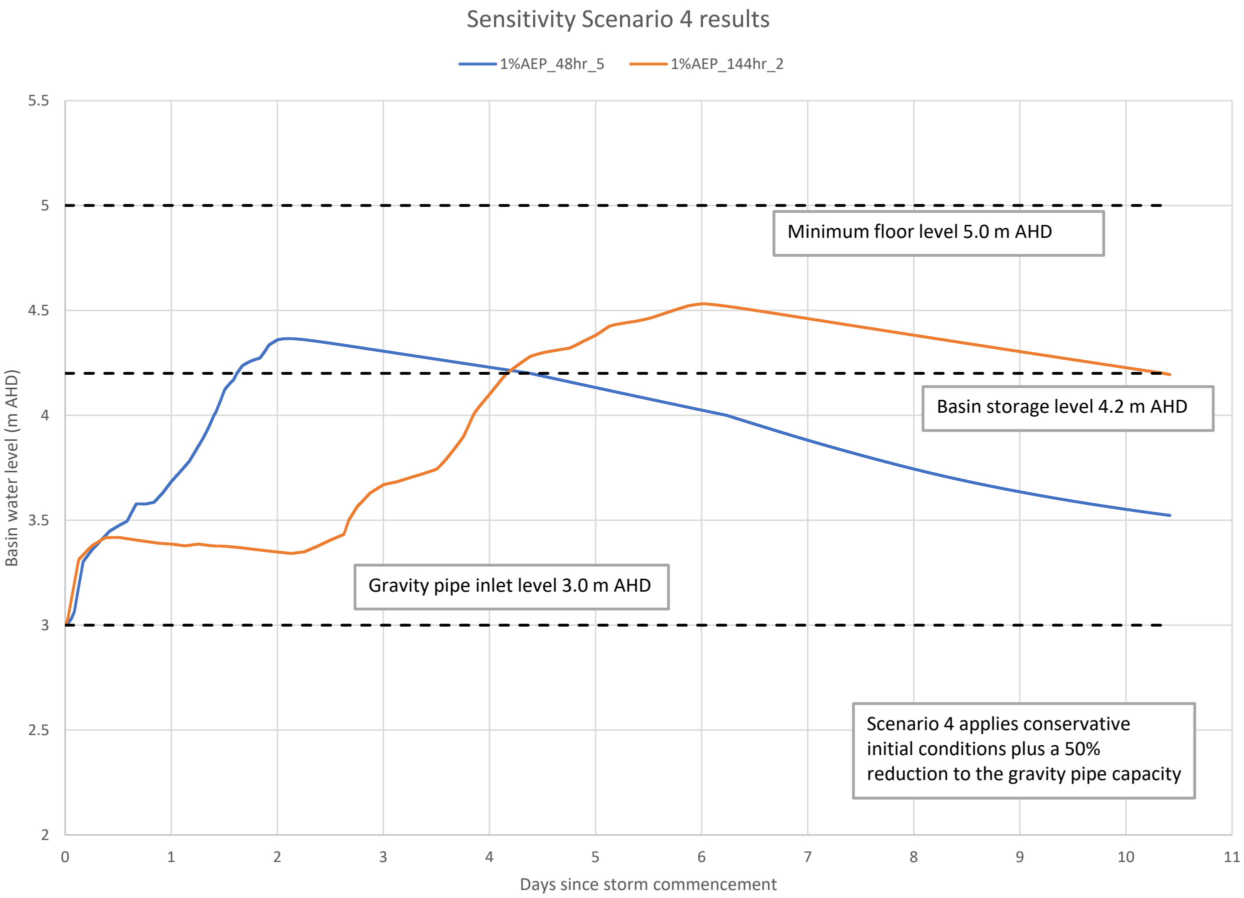

Figure C.4 Basin water level hydrographs – 1% AEP event: Sensitivity Scenario 4

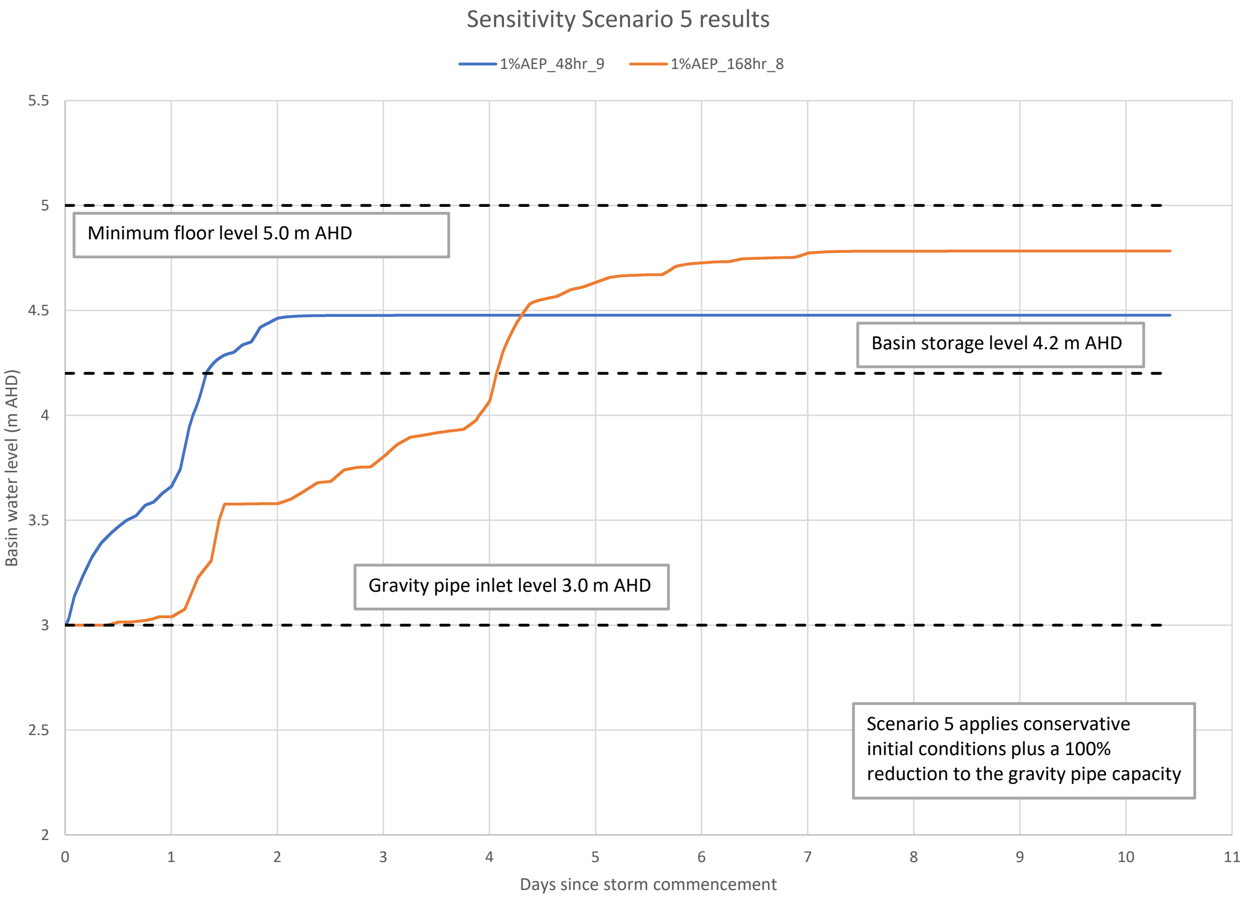

Figure C.5 Basin water level hydrographs – 1% AEP event: Sensitivity Scenario 5

Appendix D

AIP assessment framework

Department of Primary Industries
Office of Water

AQUIFER INTERFERENCE ASSESSMENT FRAMEWORK

Assessing a proposal against the NSW Aquifer Interference Policy – step by step guide

Note for proponents

This is the basic framework which the NSW Office of Water uses to assess project proposals against the **NSW Aquifer Interference Policy (AIP).**

The NSW Aquifer Interference Policy can be downloaded from the NSW Office of Water website (www.water.nsw.gov.au under Water management > Law and policy > Key policies > Aquifer interference).

While you are not required to use this framework, you may find it a useful tool to aid the development of a proposal or an **Environmental Impact Statement (EIS).**

We suggest that you summarise your response to each AIP requirement in the tables following and provide a reference to the section of your EIS that addresses that particular requirement. Using this tool can help to ensure that all necessary factors are considered, and will help you understand the requirements of the AIP.

Table 1. Does the activity require detailed assessment under the AIP?

	Consideration	Response
1	Is the activity defined as an aquifer interference activity?	Yes
2	Is the activity a defined minimal impact aquifer interference activity according to section 3.3 of the AIP?	Yes

Note for proponents

Section 3.2 of the AIP defines the framework for assessing impacts. These are addressed here under the following headings:

1. Accounting for or preventing the take of water
2. Addressing the minimal impact considerations
3. Proposed remedial actions where impacts are greater than predicted.

1. Accounting for, or preventing the take of water

Where a proposed activity will take water, adequate arrangements must be in place to account for this water. It is the proponent's responsibility to ensure that the necessary licences are held. These requirements are detailed in Section 2 of the AIP, with the specific considerations in Section 2.1 addressed systematically below.

Where a proponent is unable to demonstrate that they will be able to meet the requirements for the licensing of the take of water, consideration should be given to modification of the proposal to prevent the take of water.

Table 2. Has the proponent:

	AIP requirement	Proponent response	NSW Office of Water comment
1	Described the water source(s) the activity will take water from?	Water Sharing Plan (WSP) for the North Coast Coastal Sands Groundwater Sources 2016, the Great Lakes Coastal Sands Groundwater Source	
2	Predicted the total amount of water that will be taken from each connected groundwater or surface water source on an annual basis as a result of the activity?	Refer to Addendum report Table 5.2 (67 to 86 ML/year estimated)	
3	Predicted the total amount of water that will be taken from each connected groundwater or surface water source after the closure of the activity?	Not applicable, the project is a residential development that will operate for perpetuity.	
4	Made these predictions in accordance with Section 3.2.3 of the AIP? (refer to Table 3, below)	Yes. Baseline groundwater conditions established in Groundwater Modelling Technical Report Section 2 Licensing conditions/rules followed (refer Addendum report Table 5.2). Minimal predicted impacts to landholders, licensed water users, GDEs or the environment (Addendum report Table 5.3)	
5	Described how and in what proportions this take will be assigned to the affected aquifers and connected surface water sources?	Refer Addendum report Section 5.3	
6	Described how any licence exemptions might apply?	Refer Addendum report Table 5.2	
7	Described the characteristics of the water requirements?	Not applicable	

	AIP requirement	Proponent response	NSW Office of Water comment
8	Determined if there are sufficient water entitlements and water allocations that are able to be obtained for the activity?	The proposed water licencing approach is discussed in Addendum report Table 5.2	
9	Considered the rules of the relevant water sharing plan and if it can meet these rules?	Project meets the rules of relevant water sharing plans.	
10	Determined how it will obtain the required water?	Refer Addendum report Section 5.3	
11	Considered the effect that activation of existing entitlement may have on future available water determinations?	The WSP states that the LTAAEL is 16,000 ML/yr (but could be increased to 23,650ML/yr). The NSW Water register shows that existing WALs have a total share of 2,148 ML, indicating that there is considerable market depth.	
12	Considered actions required both during and post-closure to minimize the risk of inflows to a mine void as a result of flooding?	Not applicable	
13	Developed a strategy to account for any water taken beyond the life of the operation of the project?	Not applicable	
Will uncertainty in the predicted inflows have a significant impact on the environment or other authorised water users? No, as the project will increase the volume of water recharged into the groundwater system, and will therefore have a neutral to beneficial impact on local groundwater levels and groundwater availability If **YES,** items 14-16 must be addressed.			
14	Considered any potential for causing or enhancing hydraulic connections, and quantified the risk?		
15	Quantified any other uncertainties in the groundwater or surface water impact modelling conducted for the activity?		

	AIP requirement	Proponent response	NSW Office of Water comment
16	Considered strategies for monitoring actual and reassessing any predicted take of water throughout the life of the project, and how these requirements will be accounted for?		

Table 3. Determining water predictions in accordance with Section 3.2.3 (complete one row only – consider both during and following completion of activity)

	AIP requirement	Proponent response	NSW Office of Water comment
1	**For the Gateway process**, is the estimate based on a simple modelling platform, using suitable baseline data, that is, fit-for-purpose?	**Yes** Refer to Groundwater Modelling Technical Report Section 6 for information on model confidence level classification.	
2	**For State Significant Development or mining or coal seam gas production**, is the estimate based on a complex modelling platform that is: • Calibrated against suitable baseline data, and in the case of a **reliable water source,** over at least two years? • Consistent with the Australian Modelling Guidelines? • Independently reviewed, robust and reliable, and deemed fit-for-purpose?	Not applicable.	
3	In all other processes, estimate based on a desk-top analysis that is: • Developed using the available baseline data that has been collected at an appropriate frequency and scale; and • Fit-for-purpose?	Not applicable.	

Other requirements to be reported on under Section 3.2.3

Table 4. Has the proponent provided details on:

	AIP requirement	Proponent response	NSW Office of Water comment
1	Establishment of baseline groundwater conditions?	Refer to Groundwater Modelling Technical Report Section 2	
2	A strategy for complying with any water access rules?	Refer to Addendum report Section 5.3	
3	Potential water level, quality or pressure drawdown impacts on nearby basic landholder rights water users?	Refer to Groundwater Modelling Technical Report Section 7 (flooding) and 8 (non flooding) A summary is provided in Addendum report Table 5-3	
4	Potential water level, quality or pressure drawdown impacts on nearby licensed water users in connected groundwater and surface water sources?	Refer to Groundwater Modelling Technical Report Section 7 (flooding) and 8 (non flooding) A summary is provided in Addendum report Table 5-3	
5	Potential water level, quality or pressure drawdown impacts on groundwater dependent ecosystems?	Refer to Addendum report Table 5-3	
6	Potential for increased saline or contaminated water inflows to aquifers and highly connected river systems?	Addendum report Table 5-3	
7	Potential to cause or enhance hydraulic connection between aquifers?	All excavations will be undertaken within the upper portion of the coastal sands aquifer. Hence, there is no potential to cause or enhance a connection with another aquifer.	
8	Potential for river bank instability, or high wall instability or failure to occur?	There is no potential to impact river bank or high wall instability.	
9	Details of the method for disposing of extracted activities (for coal seam gas activities)?	Not applicable.	

2. Addressing the minimal impact considerations

Note for proponents

Section 3.2.1 of the AIP describes how aquifer impact assessment should be undertaken.

1. Identify all water sources that will be impacted, referring to the water sources defined in the relevant water sharing plan(s). Assessment against the minimal impact considerations of the AIP should be undertaken for each ground water source.
2. Determine if each water source is defined as 'highly productive' or 'less productive'. If the water source is named in then it is defined as highly productive, all other water sources are defined as less productive.
3. With reference to pages 13-14 of the Aquifer Interference Policy, determine the sub-grouping of each water source (eg alluvial, porous rock, fractured rock, coastal sands).
4. Determine whether the predicted impacts fall within Level 1 or Level 2 of the minimal impact considerations defined in Table 1 of the AIP, for each water source, for each of water table, water pressure, and water quality attributes. The tables below may assist with the assessment. There is a separate table for each sub-grouping of water source – only use the tables that apply to the water source(s) you are assessing, and delete the others.
5. If unable to determine any of these impacts, identify what further information will be required to make this assessment.
6. Where the assessment determines that the impacts fall within the Level 1 impacts, the assessment should be 'Level 1 – Acceptable'
7. Where the assessment falls outside the Level 1 impacts, the assessment should be 'Level 2'. The assessment should further note the reasons the assessment is Level 2, and any additional requirements that are triggered by falling into Level 2.
8. If water table or water pressure assessment is not applicable due to the nature of the water source, the assessment should be recorded as 'N/A – reason for N/A'.

Table 5. Minimal impact considerations

Aquifer	Coastal sands
Category	Highly productive
Level 1 Minimal Impact Consideration	**Assessment**
Water table Less than or equal to a 10% cumulative variation in the water table, allowing for typical climatic 'post-water sharing plan' variations, 40 metres from any: • high priority groundwater dependent ecosystem or • high priority culturally significant site listed in the schedule of the relevant water sharing plan. **OR** A maximum of a 2 metre water table decline cumulatively at any water supply work.	Addendum report Table 5.3 – minimal impact
Water pressure A cumulative pressure head decline of not more than a 2 metre decline, at any water supply work.	Addendum report Table 5.3 9 - minimal impact
Water quality Any change in the groundwater quality should not lower the beneficial use category of the groundwater source beyond 40 metres from the activity.	Addendum report Table 5.3 – minimal impact

3. Proposed remedial actions where impacts are greater than predicted.

Note for proponents

Point 3 of section 3.2 of the AIP provides a basic framework for considerations to consider when assessing a proponent's proposed remedial actions.

Table 6. Has the proponent:

AIP requirement		Proponent response	NSW Office of Water comment
1	Considered types, scale, and likelihood of unforeseen impacts *during operation*?	A comprehensive assessment of the water management constraints and opportunities was undertaken based on available data. The proposed surface and groundwater management strategy was formulated to respond to identified constraints and risks.	
2	Considered types, scale, and likelihood of unforeseen impacts *post closure*?	Not applicable, the project is a residential development that will operate for perpetuity	
3	Proposed mitigation, prevention or avoidance strategies for each of these potential impacts?	A comprehensive assessment of the water management constraints and opportunities was undertaken based on available data. The proposed surface and groundwater management strategy was formulated to respond to identified constraints and risks. Potential impacts are considered minimal.	
4	Proposed remedial actions should the risk minimization strategies fail?	No remedial actions are proposed. Potential impacts are considered minimal. However, as discussed below there is flexibility in the design of the open basins.	
5	Considered what further mitigation, prevention, avoidance or remedial actions might be required?	The open basins can be filled to become ephemeral basins if required. This would not impact the functionality of the water management system.	
6	Considered what conditions might be appropriate?	The final form of the open basins will be negotiated with Council and other government stakeholders.	

4. Other considerations

Note for proponents

These considerations are not included in the assessment framework outlined within the AIP, however are discussed elsewhere in the document and are useful considerations when assessing a proposal.

Table 7: Has the proponent:

	AIP requirement	Proponent response	NSW Office of Water comment
1	Addressed how it will measure and monitor volumetric take? (page 4 of the AIP)	Monitoring of water take for open space irrigation will be undertaken using a standard method such as a flow meter. Volumetric water take from the basins will be established by calculation if required to be licenced.	
2	Outlined a reporting framework for volumetric take? (page 4 of the AIP)	Reporting of water take for open space irrigation will be undertaken using standard methods.	

More information

www.water.nsw.gov.au

Disclaimer:

This is a draft document produced as a guide for discussion, and to aid interpretation and application of the NSW Aquifer Interference Policy (2012). All information in this document is drawn from that policy, and where there is any inconsistency, the policy prevails over anything contained in this document.
Any omissions from this framework do not remove the need to meet any other requirements listed under the Policy.

The information contained in this publication is based on knowledge and understanding at the time of writing (March 2021). However, because of advances in knowledge, users are reminded of the need to ensure that information upon which they rely is up to date and to check currency of the information with the appropriate officer of the Department of Primary Industries or the users independent adviser.

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