Stormwater Management Plan

Gunnedah Saleyards

Gunnedah Shire Council

11 June 2021

→ The Power of Commitment

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1. Introduction

1.1 Background

A requirement of the Gunnedah Saleyards Development Application was to prepare a Stormwater Management Plan (SWMP) to determine the impact to the sites existing drainage system. GHD was commissioned by Gunnedah Shire Council (Council) to develop an SWMP for the purposes of assessing the proposed design of a new site entrance and infrastructure at the Gunnedah Saleyards and associated impact to the stormwater drainage system for the site.

This SWMP documents the:

– Existing site drainage and flow
– Proposed development and resulting flow
– Stormwater storage tank water balance
– Recommendations for compliance with Council specifications

1.2 Purpose of this report

The objective of this SWMP is to assess the resulting flows from the proposed upgrades at the Gunnedah Saleyards and demonstrate these flows can be intercepted, drained and lawfully discharged in the designed storm events as to not create nuisance to any downstream neighbours. This includes assessing proposed implementation of detention to ensure post development flows do not exceed pre-development flows from site. The report also documents the process for determining the size of rainwater tanks which capture water from the proposed roof structures over the lots and building.

1.3 Scope and limitations

1.3.1 Scope of services

The development of this SWMP included the following scope of services:

Stormwater quantity assessment

– Obtain flood data for the site (to determine whether the site is within the floodplain or whether consideration of tailwater conditions is required).
– Delineate internal catchments.
– Selection of catchment parameters (area, fraction impervious, time of concentration and slope).
– Identify overland flow paths.
– Develop a DRAINS model to assess the stormwater drainage network for stormwater discharging onto the site in 20 year and 100 year ARI rainfall events.
– Propose a stormwater drainage (major and minor flow) strategy for the proposed development. Note that detailed design is not part of the SWMP, only recommendations into mitigation/detention solutions, if required.

Rainwater tank calculations

– Undertake a water balance as`sessment to determine the require volume of rainwater storage for use within the site. The assessment is to be based on water meter data provided by Council and historical rainfall data.
– Recommendations on rainwater tank dimensions is to be used in the developed scenario DRAINS model for use in detention estimates.

Stormwater quality management

– Not required as part of this Stormwater Management Plan.

Data collection

The following data was used to undertake this project:

- Discussion with Council:
  - Site induction meeting held with Gunnedah Shire Council and GHD's Project Manager John Roworth
  - Email correspondence between GHD and Council engineers
- Data supplied by Council:
  - Extract of stormwater DA conditions from Council's DA
  - LiDAR data (2014)
  - GIS servicing data (2021)
  - Original site survey by Stewart Surveys (2021)
  - Original DA survey by Kelley Covey (2021)
  - Site development layout plan for DA by Studio Two Architecture (2021)
  - Site roofing plan for DA by Hill Lockart Architects (2021)
  - Pump data curves for existing sump pump (2006)
- Aerial photography from Google Earth (2021)
- Additional site survey (incl. service locating) carried out by RPS Group (2021)
- Bureau of Meteorology Intensity-Frequency-Duration (IFD) data (2021)
- Australian Rainfall and Runoff Data Hub (2021)

1.3.2 Limitations

This report: has been prepared by GHD for Gunnedah Shire Council and may only be used and relied on by Gunnedah Shire Council for the purpose agreed between GHD and Gunnedah Shire Council as set out in Section 1.2 of this report.

GHD otherwise disclaims responsibility to any person other than Gunnedah Shire Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer Sections 3.2 and 4.1.2 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the location of buildings, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this report.

Site conditions (including the presence of hazardous substances and/or site contamination) may change after the date of this Report. GHD does not accept responsibility arising from, or in connection with, any change to the site conditions. GHD is also not responsible for updating this report if the site conditions change.

GHD has prepared this report on the basis of information provided by Gunnedah Shire Council and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

Any reports, drawings, memos, or other deliverables produced by GHD shall be produced in a traditional and generally accepted format. Accessible reports, drawings, memos, or other deliverables can be provided by GHD at an additional cost if necessary.

2. Site description

2.1 Location

Gunnedah Saleyards are located along the Kamilaroi Highway, approximately 1.3 km west of Gunnedah's main town centre, with the Namoi River travelling approximately 500 m north of the site, see Figure 2.1 below.

Figure 2.1 Regional site location

Figure 2.2 General site location

The site is bounded to the northwest by Council's Water Treatment Plant and by the Kamilaroi Highway to the south, see Figure 2.2. The site is surrounded by greenspace directly to the north and east, through a tributary of the Namoi River. A small portion of Gunnedah's low density urban residential developments are situated to the east of the site, which are unaffected by any surface flow from the saleyards as the site grades generally to the north.

2.2 Rainfall and climate

Rainfall data for the Gunnedah area was obtained from the Australian Rainfall & Runoff (ARR) Data Hub and is tabulated in Appendix A. This data was used to model all rainfall events to assess existing and post-development stormwater management.

ARR considers Gunnedah a 'semi-arid inland' region which is defined as a dry climate with low average rainfall of low intensities. Data from the Bureau of Meteorology shows the highest daily rainfall in the region since 2007 was in the 24 hours leading up to 9 am on 29 November 2008 with 111.6 mm.

2.3 Site survey

The site was originally surveyed by Stewart Surveys (2021), with an updated survey carried out by Kelley Covey later in 2021 to detail Council's DA proposed layout. GHD have recently engaged RPS Group as part of these works for additional survey to "fill the gaps" from previous surveys and to accurately locate underground services to assist in detailed design of the proposed development. This additional survey was used for catchment delineation for the purpose of this SWMP.

2.4 External flooding

The site is adjacent to the confluence of Blackjack Creek and the Namoi River. A flood study for Blackjack Creek was completed in 2005 for Council by Lyall & Associates, however, the assessment boundary is upstream of the project site and flood mapping does not extend to the site. Flood mapping for Gunnedah which was created for of the Gunnedah Floodplain Management Plan (SMEC, 2009) indicates that the project site is impacted by the Namoi River regional catchment for the 5% AEP (approximately 20 year ARI) and above. Figure 2.3 below provides the flood mapping for the 1% AEP extents for the site, with the approximate site boundary shown in red.

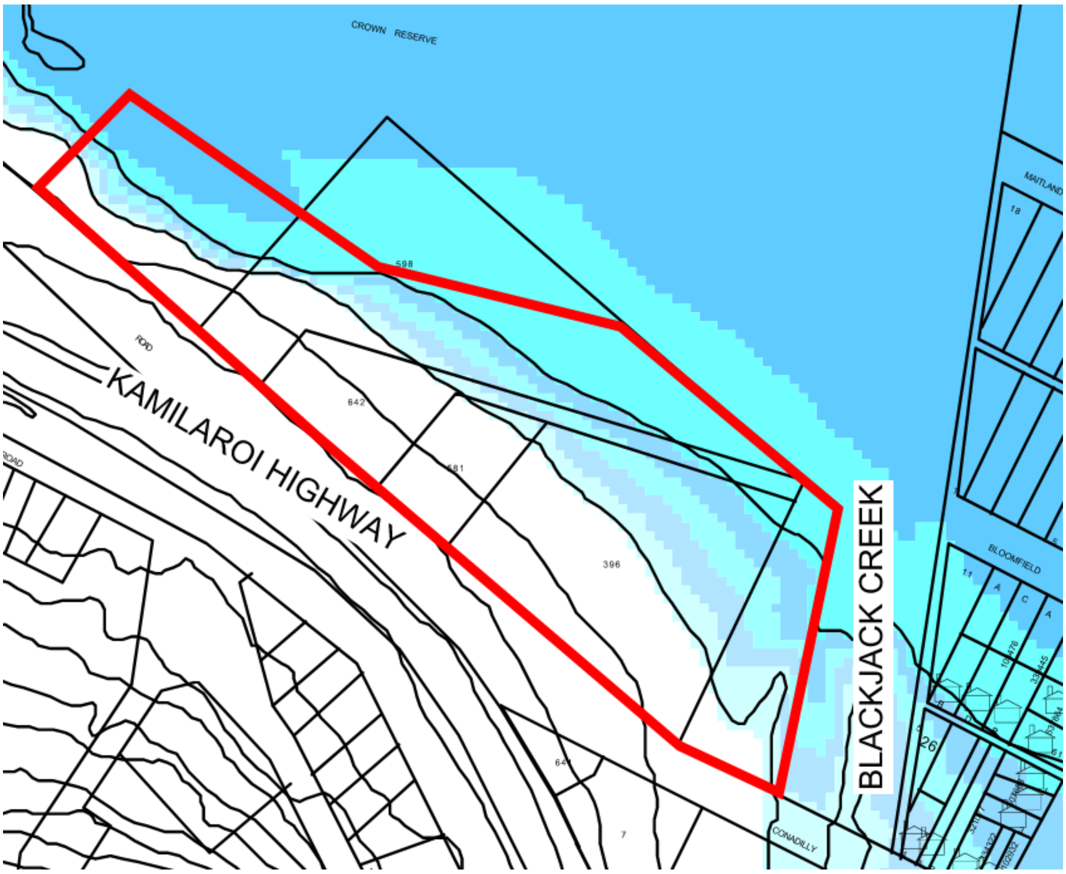


Figure 2.3 Gunnedah Floodplain 1%AEP extents at project site

Refer to Appendix B for the 1%AEP flood mapping which was completed for the Gunnedah Floodplain Management plan. Note that the mapping does not appear to include flooding from Blackjack Creek, only the Namoi River.

2.5 Existing drainage strategy

The existing site drainage strategy is comprised of the following (refer to Figure 2.4 for the existing site drainage):

– A hardstand area (truck wash bay) which flows through a culvert directly into a treatment pond (Pond 1).
– Overland flow across the western portion of site which flows directly into either treatment pond (Pond 1 or 2).
– Overland flow across the eastern portion of site which flows north into a graded channel.
– A second hardstand area which collects the overland flow from the graded channel.
– A sump which collects water from the second hardstand area and pumps it into a treatment pond (Pond 1).
– An overflow culvert from the first treatment pond (Pond 1) into a second treatment pond (Pond 2).
– An overflow culvert from the last treatment pond (Pond 2) into GSC Sewerage Treatment Dam.

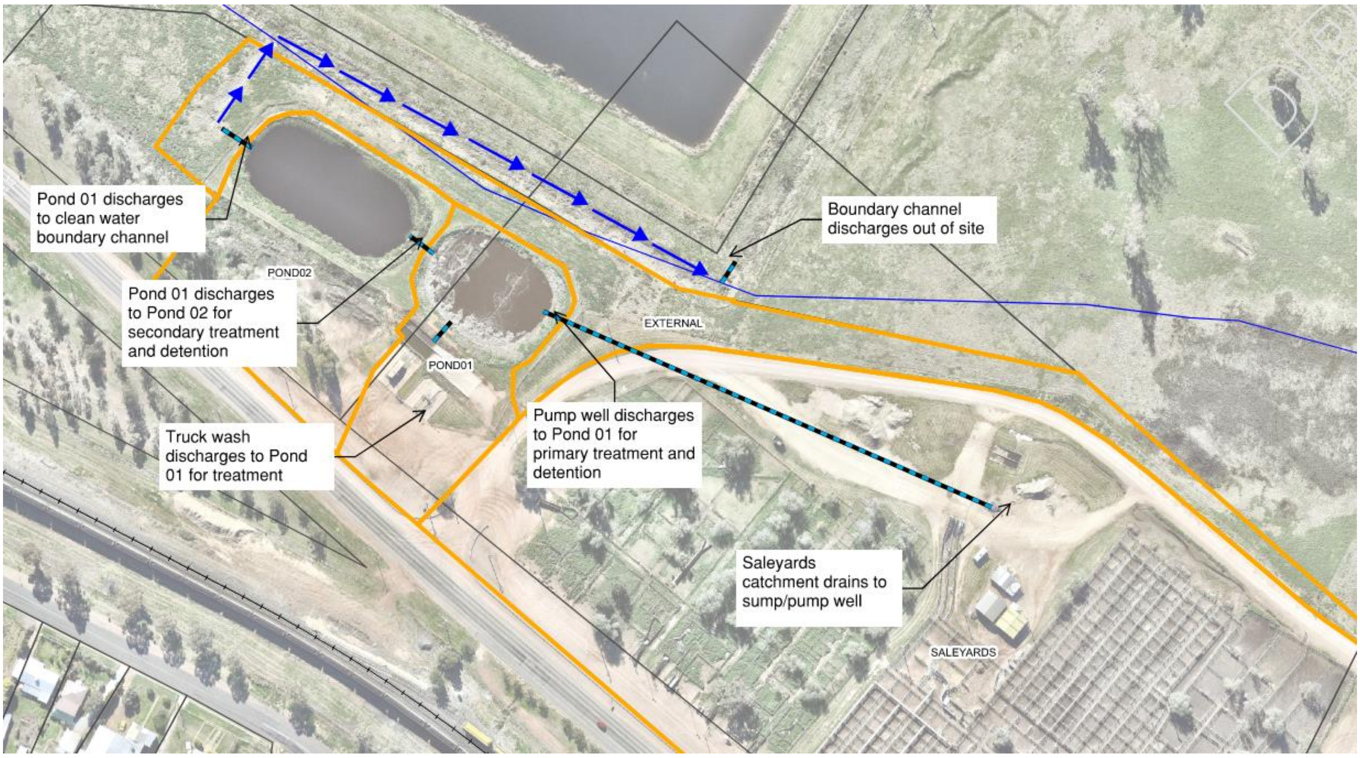


Figure 2.4 Existing site drainage

Existing sump and pump well

The existing sump associated with the second hardstand area captures all runoff from within the site where stock are transported or stored. Council have provided information on the structure and pump which appears to be approximately 2.2 m internal diameter, however, the depth is unknown. Based on discussions with Council, the depth of the pump well is approximately 5 m below the existing ground level, or 4 m below the lowest point of the washout bay.

The documentation provided indicates that the duty point for the pump is 8 m, which corresponds with a flow rate of 55 l/s.

Council have noted this sump has only overtopped once since it’s commissioning in 2007, however, the date of this event is unknown.

Figure 2.5 and Figure 2.6 below provides a plan and section of the sump arrangement.

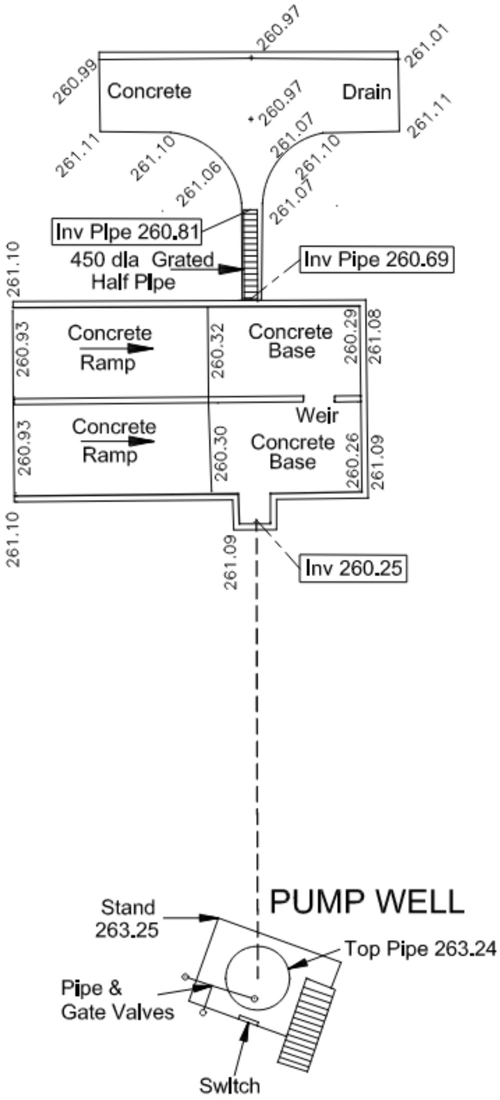


Figure 2.5 Washout bay and pump well plan from survey

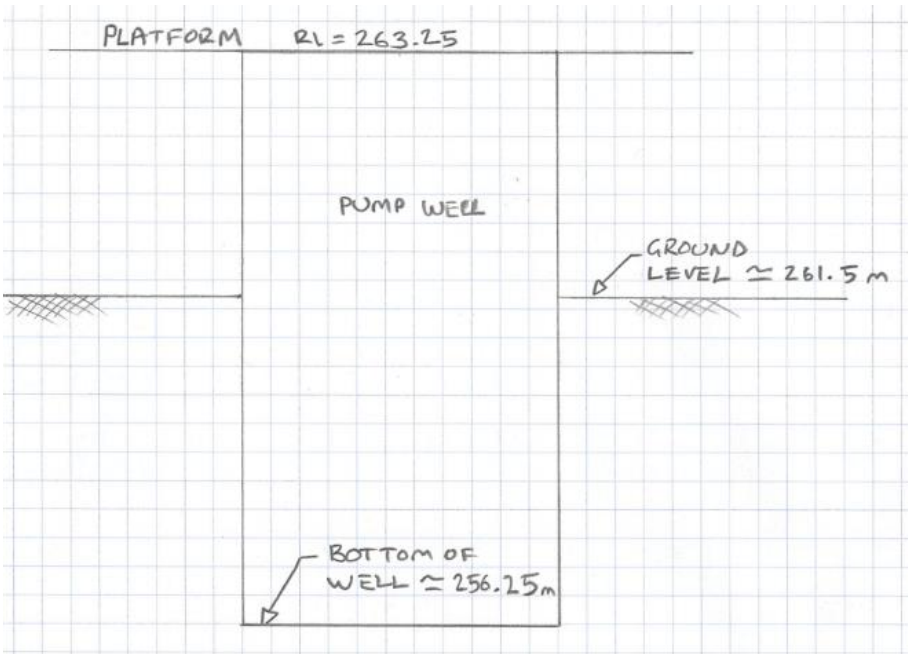


Figure 2.6 Sketch of assumed pump well levels

3. Water balance assessment

3.1 Overview

A water balance was prepared for the proposed runoff collection and reuse system. This assessment was undertaken to determine the range of potential storage options at the site and to assess the proportion of time that demand at the site will be met, and operating deficits.

3.2 Methodology

3.2.1 Water balance structure

A simple water balance structure was utilised for this assessment and is described below:

– Daily rainfall on roof structures and hardstands are converted into runoff assuming a volumetric coefficient of 1.0. Given a total catchment of 25,567 square metres, 1 mm of rainfall will result in approximately 25.6 m^3 (or kL of runoff). Catchment delineation is shown below in Table 3.1 and Figure 3.1.
– Water is stored in tanks up to a maximum volume, where it is assumed to be lost and spilled.
– Water is extracted from the tanks to meet demand every Tuesday, coinciding with days that the saleyards are open. It is assumed that 100% of weekly demand occurs on a Tuesday.
– External water sources (e.g., potable, recycled) is not used to fill up the tanks, rather they are utilised directly from the source.
– Demand is based on meter information provided by Council, including meters used prior to 2019 (06HC04471 and WST0802902), and meters used after 2019 (18HC00327 and 19W711750). Weekly usage was estimated as being 236.9 kL/week prior to mid-2019 and 147.3 kL/week after mid-2019. These two demand options were both simulated, including a third option based on an average demand of 192.1 kL/week.
– No system losses from the storage (evaporation, leakage, etc.) have been assumed.

Table 3.1 Delineation of catchments for water balance assessment

Area	Volumetric runoff coefficient	Catchment (square meters)
Roof A	1.0	6,330
Roof B	1.0	9,850
Roof C1	1.0	3,790
Roof C2	1.0	3,790
Roof D and Proposed Building	1.0	1,807
Total	**1.0**	**25,567**

Figure 3.1 Catchment delineation (collected portions)

3.2.2 Climate inputs

Climatic rainfall data for this region was extracted from SILO Long Paddock[1] from the period from 1 January 1900 until 31 December 2020 (120-years total). Daily data was extracted for rainfall in a point format. The point format data for the Gunnedah region was extracted as a time series, based upon interpolating observations from station records around Australia. Station point datasets are available at approximately 8,000 station locations around Australia. A summary of Gunnedah's monthly rainfall is shown below in Figure 3.2.

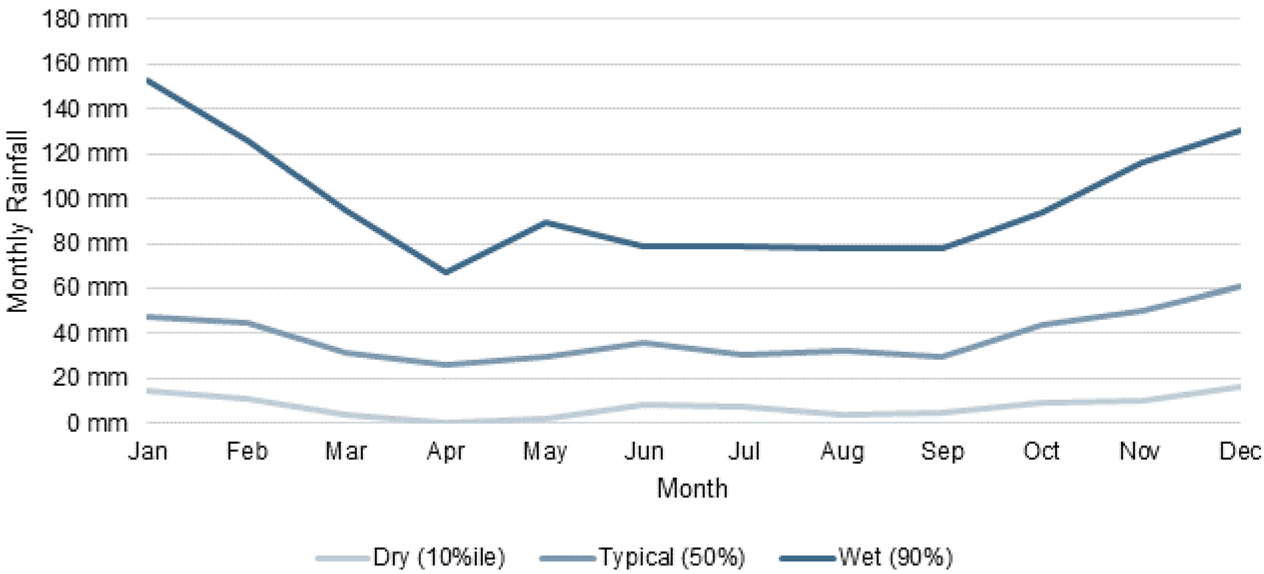


Figure 3.2 Monthly rainfall statistics from SILO

[1] https://www.longpaddock.qld.gov.au/silo/point-data/

3.2.3 Simulation

Simulation of the water balance was undertaken using Microsoft excel, on a daily timestep. Simulation was undertaken for the full climatic period of 120 years, from 1 January 1900 until 31 December 2020 based upon the proposed system configuration. A range of storage options were evaluated ranging from 0 kL of total storage, up to 2,000 kL (2 ML) of total storage as well as three weekly demands of 147.3 kL/week, 192.1 kL/week and 236.9 kL/week.

3.3 Demand assessment

Metrics from the water balance model were utilised to assess potential storage options based on two criteria:

1. The long-term proportion of time that the demand can be met from on-site storage. This metric of reliability increases with an increase in total storage volume.
2. The long-term average weekly deficit of water. This is the additional water necessary to fully meet the sites demand. This metric of external reliance decreases with an increase in total storage volume.

Results of a range of storage options are shown below Figure 3.3. The results indicate that:

– Considerable onsite storage is necessary to meet demand at the site. At a minimum, 147 – 240 kL of storage is necessary to be able to provide a single week's demand.

– To provide 50% reliability, a storage in the range of 300 – 500 kL is necessary.

– To achieve 90% reliability, total storage would need to be at least 900 kL for a demand of 147.3 kL. It is noted that the ability to achieve this reliability may not be pragmatic for higher demands, considering the rapid increase of storage required to provide this higher reliability.

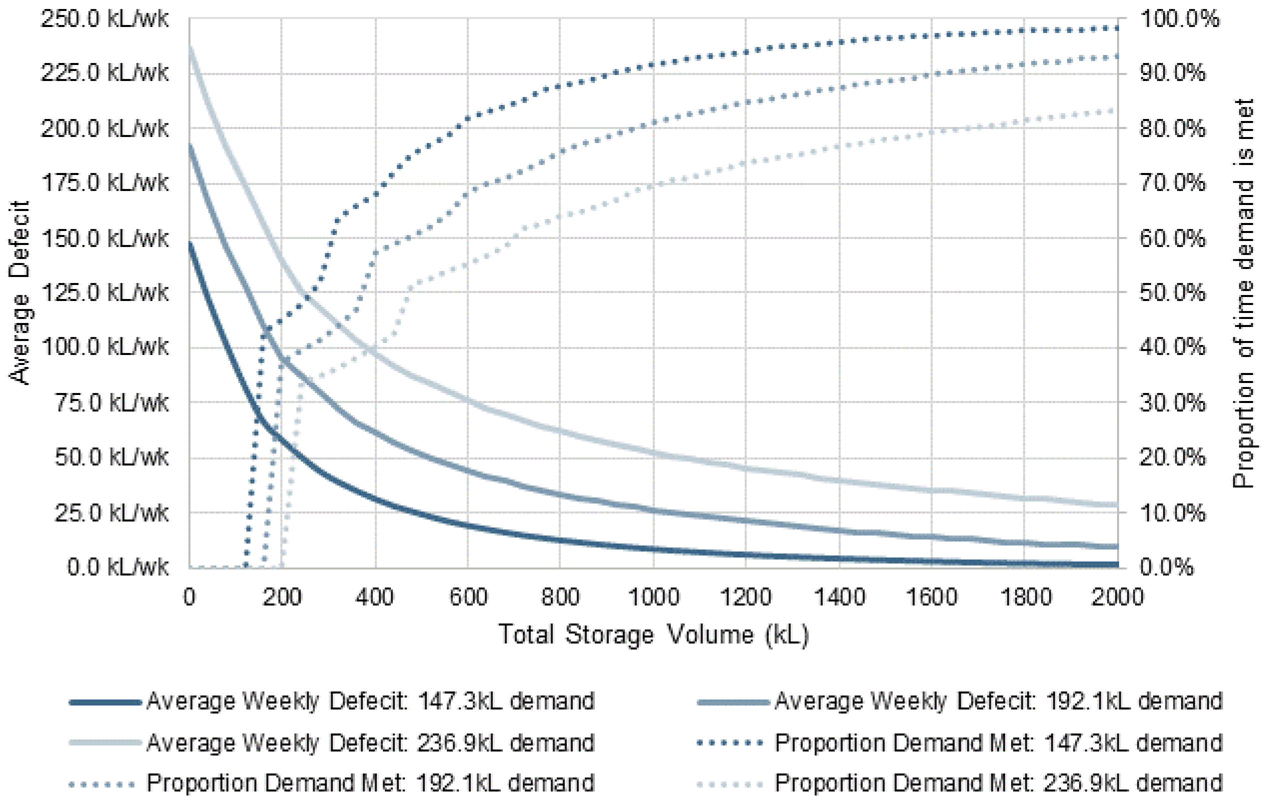


Figure 3.3 Water balance simulation results

4. Stormwater quantity assessment

4.1 Overview

4.1.1 General

The stormwater quantity assessment involved the following components:

– Delineation of existing catchments and identification of existing overland flow paths.
– Delineation of catchments resulting from the proposed development and identification of resulting overland flow paths.
– Development of a DRAINS model to assess the existing pre-development stormwater drainage network.
– Development of a DRAINS model to assess the proposed post-development stormwater drainage network.
– Assessment of the increase in stormwater quantity to the existing outlet as a result of the proposed development.
– Proposal of a stormwater drainage strategy including appropriate storage and detention.

4.1.2 Assumptions

The following assumptions were adopted when constructing the stormwater quantity assessment:

– Site catchments were delineated based on existing and recent site survey data.
– Proposed site catchments were delineated based on the concept design provided. Detailed drainage design has not been considered for this stage of the project.
– Overland flow paths and discharge locations were based on the survey provided and discussions with Council.
– The pump curve data provided by Council relates the assumed 8 m of head with a design flow rate of 55 L/s which is used in calculations. A constant pump flow rate of 55 L/s has been adopted for modelling purposes.

4.1.3 Stormwater drainage design guidelines and requirements

The guideline used for the SWMP was the Gunnedah Shire Council Engineering Guidelines for Subdivisions & Developments, effective August 2013. Section 3 – Guidelines for Stormwater Drainage Design details Councils requirements for the minor and major system design approach.

Council’s DA conditions require the minor system to be designed for the 5% AEP (approximately 1 in 20 year ARI) in accordance with ARR and the major system to be designed for the 1% AEP (1 in 100 year ARI).

4.2 Methodology

4.2.1 Existing catchments

A breakdown of the existing catchments which were delineated based on survey data are detailed below in Table 4.1. These values were used to calculate the pre development flows in the existing scenario DRAINS model. Figure 4.1 below shows the layout of these existing catchments and the impervious (grey) and pervious (green) areas within the project area.

In accordance with ARR19, the effective impervious area was adopted for the saleyards catchment. Although the majority of the catchment has been identified as impervious due to heavily compacted roads and yards, there will still be infiltration in these areas as well as the vegetated areas. The Total Impervious Area (TIA) area was calculated using GIS software to be 71% which was then factored by 65% to account for the additional losses expected, resulting in an Estimated Impervious Area (EIA) of 45% of the catchment. The EIA was not considered for the other catchments due to their small size. The ‘external’ catchment was not included in the model as it does not change as part of the development and only conveys clean runoff.

Table 4.1 Existing catchments

Catchment name	Area (ha)	% Impervious	Comment
SALEYARDS	7.2	45	Effective impervious area
POND01	0.62	70	
PODN02	0.89	37	
EXTERNAL	1	0	Not included in model

Figure 4.1 Existing catchments and impervious surfaces

4.2.2 Developed catchments

A breakdown of the proposed developed catchments which were delineated based on Council's proposed DA layout are detailed below in Table 4.2. These values were used to calculate the post development flows in the developed scenario DRAINS model. Figure 4.1 below shows the general layout of the proposed developed catchments. The proportion of impervious area will increase significantly from the existing catchments due to the significant amount of roof area proposed. The impact of this is discussed in Section 4.2.4.

Table 4.2 Existing catchments

Catchment name	Area (ha)	% Impervious	Comment
Yard roof	2.33	100	
Admin building	0.07	100	
Car park	0.03	100	
Yard road and parking	4.50	55	Total Impervious Area
Pond01	0.79	77	
Pond02	0.47	54	
External	3.21	0	Not included in model

Figure 4.2 Proposed developed catchments

4.2.3 Developed case – overland flow path

There is not a significant change to the overland flow paths as a result to the proposed development, as there is not a significant change proposed to the grading of the site itself. The existing and proposed developed overland flow paths will both be directed to the north as per the grading of the site.

The exception to existing flow paths is the proposed roofed areas (yard, carpark and admin building) are to be captured via rainwater tanks. The overflow for these tanks is to be discharged directly into Blackjack Creek to the east. The carpark pavement is also to drain directly into this network.

4.2.4 Developed case – storage and detention

The following stormwater storage aspects form part of Council's proposed development (see Figure 3.1):

– A water tank to capture roof runoff from Roof A, Roof B, Admin building and Rood D.
– A water tank to capture roof runoff from Roof C1 and Roof C2.

The overflow from these two tanks is proposed to discharge directly to Blackjack Creek. The rainwater tanks provide water suitable for vehicle washdown, irrigation, stock water and other non-potable uses. They also provide some detention during rainfall events but they have not been designed to provide complete attenuation for the 1% AEP event. This is discussed further in Section 4.3.4.

4.2.5 Design Storms

The minor and major storm events have been selected as 5% and 1% AEP events respectively, in accordance with Council's DA conditions as discussed in Section 4.1.3. Rainfall data was obtained from the Bureau of Meteorology online IFD application and the ARR Data Hub which is provided in Appendix A.

4.3 Hydrological and hydraulic assessment

4.3.1 Drains

DRAINS is a stormwater drainage system design and analysis program. It is widely used for urban stormwater system design and analysis in Australia. The DRAINS program performs hydrologic assessments, hydraulic grade line analysis, design of stormwater drainage systems and produces summary graphs and tables and pipe long section drawings.

A high-level DRAINS model was developed in order to assess the existing drainage network capacity and the impact of the proposed developments at the saleyard on the drainage network. The DRAINS model layout is simulated in an Existing vs. a Development arrangement.

Model assumptions

The following assumptions were made in the development of the DRAINS model.

– Drainage networks were simplified for existing and developed scenarios by not using lumped catchments and not incorporating pit/piped networks. This is a suitable strategy for concept level documentation.
– Roof catchments were taken as being 100% impervious.
– Effective Impervious Areas were only used for large catchments which have a mixture of impervious, grassed and compacted dirt surfaces.
– Where necessary, practical stormwater design assumptions based on best practice engineering judgement were made where the site survey did not capture the full extent of the stormwater infrastructure.

Catchment delineation

In establishing the DRAINS model, the site was divided into a number of catchments, which followed major overland flow routes and stormwater drainage trunk mains through the site.

4.3.2 Existing scenario

The existing scenario model was assessed for both the minor (5% AEP) and major (1% AEP) events. Modelling shows that for both the minor and major events, the ponds are adequately sized to provide detention without overtopping.

Although undocumented, various ARI storms were run to determine the sump performance. The model results indicate that for the 20% AEP and above, the sump/pump well is undersized. Council have confirmed that the sump has only overtopped once since it was constructed in 2007, but are unable to confirm the exact date of the storm. Bureau of Meteorology historical rainfall data indicates that since 2007 there has been one day with over 100 mm of rainfall, which was recorded in the 24 hours prior to 9 am on 29 November 2008 with a total of 111.6 mm. An analysis of detailed rainfall data reveals that this storm event varied in intensity from an 0.2EY to 2% AEP. The storm had very close correlation with the 2% AEP between the 30 minute and 2 hour durations.

The next highest daily rainfall recorded was 80 mm which occurred in the 24 hours prior to 9 am on both 29 January 2013 and 30 March 2019. Detailed data for the 30 March 2019 event was not available, and therefore only the 29 January 2013 storm event was assessed. The assessment revealed that the storm correlated with the 12EY event up to the 15 minute duration, but after that the intensities increased to align with the 0.2EY AEP event at the 12 hour duration.

The model contradicts the historical data and anecdotal evidence on flooding, however, the thresholds of the sump overtopping are similar enough that the model can be considered adequate for this level of assessment.

No other significant issues were identified for the existing drainage arrangement for the site.

Three points of discharge are investigated for the site for both the existing and developed scenarios:

- From ponds.
- From sump.
- From site (existing discharge point).

The discharge from ponds is the flow through the low-level culvert from the western pond. This is an indicator of detention within the ponds as a result of site runoff which is generally considered ‘dirty’ prior to entering the pond treatment system. Discharge from sump is considered to be the overflow from the sump, which only occurs for storm events greater than the 5% AEP. This is an indicator of whether the sump is performing as intended. We note that the design ARI for the sump has not been provided by Council.

Discharge from the site considers overflow from the pump and flow from the basins. This is not necessarily a simple addition of the two hydrographs as the peak flow due to timing of critical durations, but is a sensible indicator of the overall peak discharge from a site when considering single points of discharge.

Minor event results

Discharge from the ponds for the minor event peaks at 0.10 m^3/s with the critical duration for the storm being 12 hours. The long duration of the critical storm is consistent with the use of large basins and is expected when detention is incorporated within the system.

The model reports overflow from the sump which peaks at 0.24 m^3/s with a critical duration of 2 hours, and a peak discharge from the site of 0.31 m^3/s with a critical duration of 2 hours. Refer to Figure 4.3 for the existing minor event hydrographs.

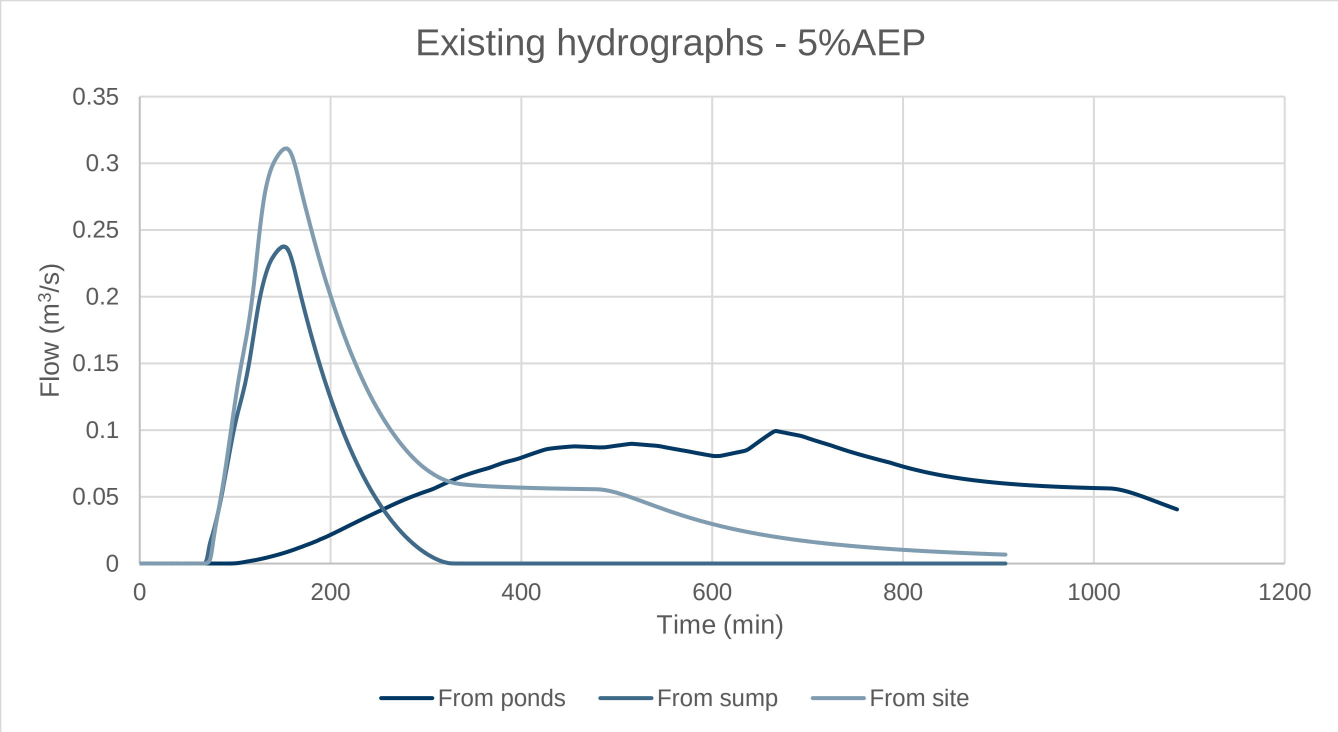


Figure 4.3 ***DRAINS model existing hydrographs – 5% AEP***

Major event results

Discharge from the ponds for the major event peaks at 0.14 m^3/s with the critical duration for the storm being 12 hours. The overflow from the sump peaks at 0.38 m^3/s with a critical duration of 2 hours. Peak discharge from the site is 0.49 m^3/s with a critical duration of 3 hours. Refer to Figure 4.4 for the existing major event hydrographs.

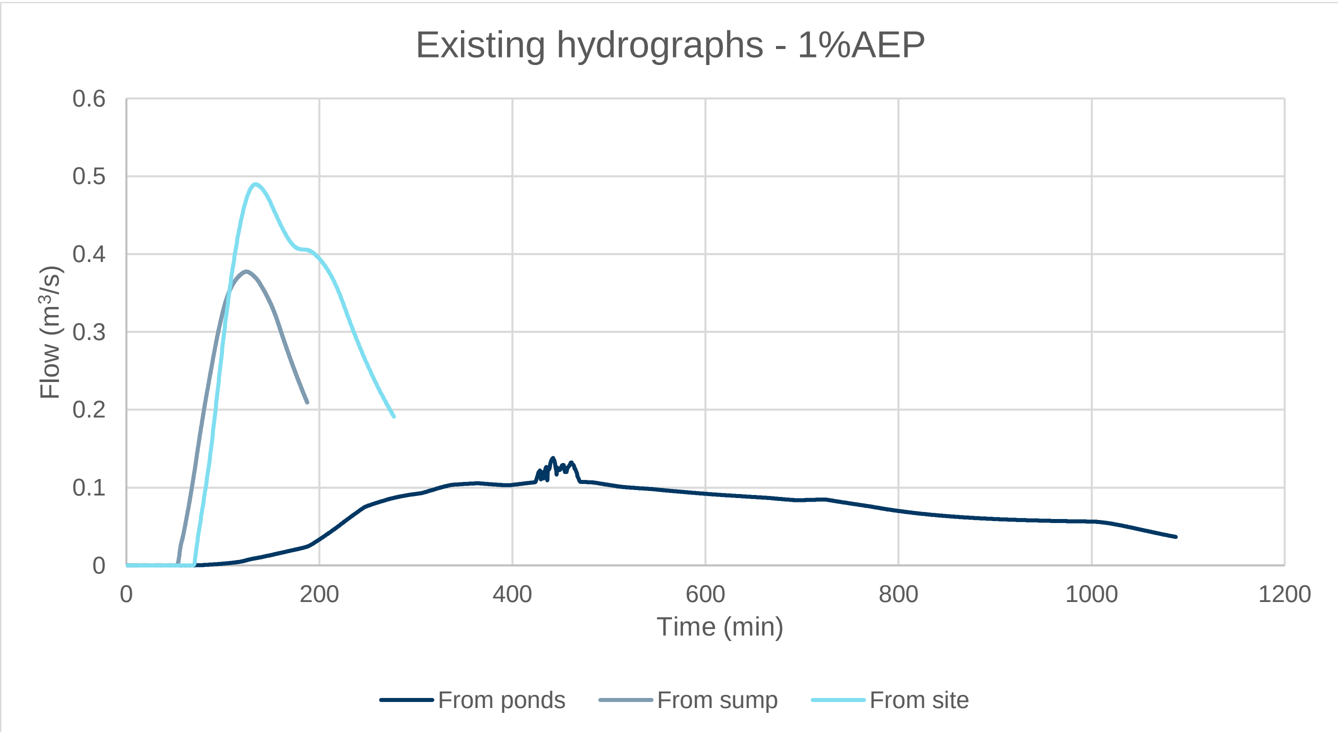


Figure 4.4 ***DRAINS model existing hydrographs – 1%AEP***

4.3.3 Developed case

The developed case DRAINS model incorporated the following changes:

- Catchments from proposed yard roofs, car park and admin building drain directly to a rainwater tank. Overflow for the rainwater tank discharges from the site at a new discharge point.
- The proposed car park drainage is captured and connected to the rainwater tank overflow.
- The catchment draining to the sump/pump well is reduced from 7.2 ha to 4.5 ha.
- The catchment draining to Pond 01 is increased from 0.62 ha to 0.79 ha.
- The initial water level for the rainwater tanks is set at the high-level outlet. This is a worst-case scenario which minimizes the amount of detention provided. The maximum height of the tanks is set to 1 m above the high level outlet.

In addition to the points of discharge identified in the existing scenario, a new point of discharge is assessed from the rainwater tanks/car park surface which is proposed at the north-eastern corner of the site. As a result, the flow from the existing discharge point is generally reduced, which is offset by the proposed discharge point.

Minor event results

Discharge from the ponds for the minor event peaks at 0.09 m^3/s with the critical duration for the storm being 6 hours. The model reports overflow from the sump which peaks at 0.14 m^3/s with a critical duration of 2 hours, and a peak discharge from the site (existing discharge point) of 0.20 m^3/s with a critical duration of 3 hours. The peak discharge from tank overflow/car park is 0.73 m^3/s with a critical duration of 15 minutes. Refer to Figure 4.5 for the proposed minor event hydrographs.

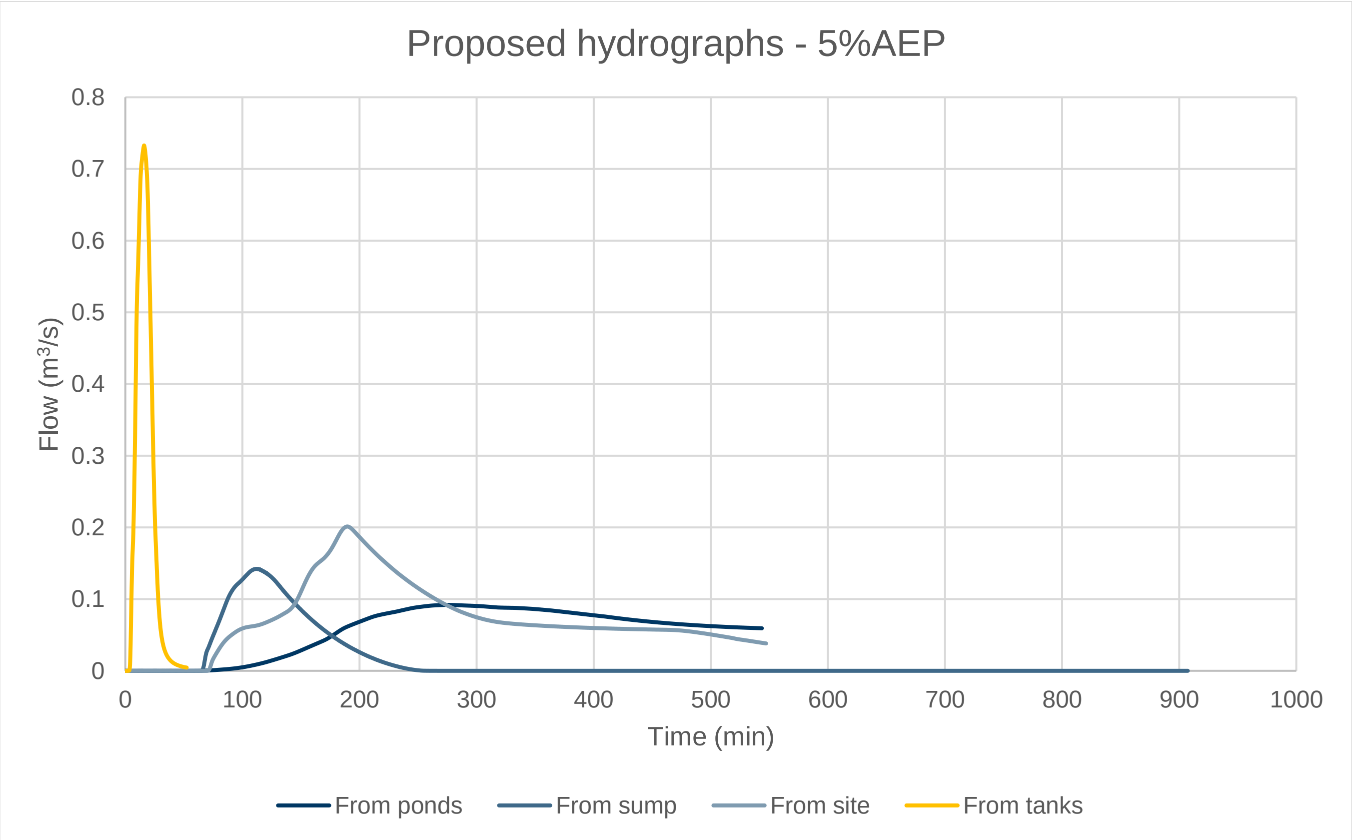


Figure 4.5 DRAINS model proposed hydrographs – 5%AEP

Major event results

Discharge from the ponds for the major event peaks at 0.10 m³/s with the critical duration for the storm being 12 hours. The overflow from the sump peaks at 0.23 m³/s with a critical duration of 2 hours, and a peak discharge from the site (existing discharge point) of 0.31 m³/s with a critical duration of 3 hours. The peak discharge from tank overflow/car park is 0.94 m³/s with a critical duration of 15 minutes. Refer to Figure 4.6 for the proposed major event hydrographs.

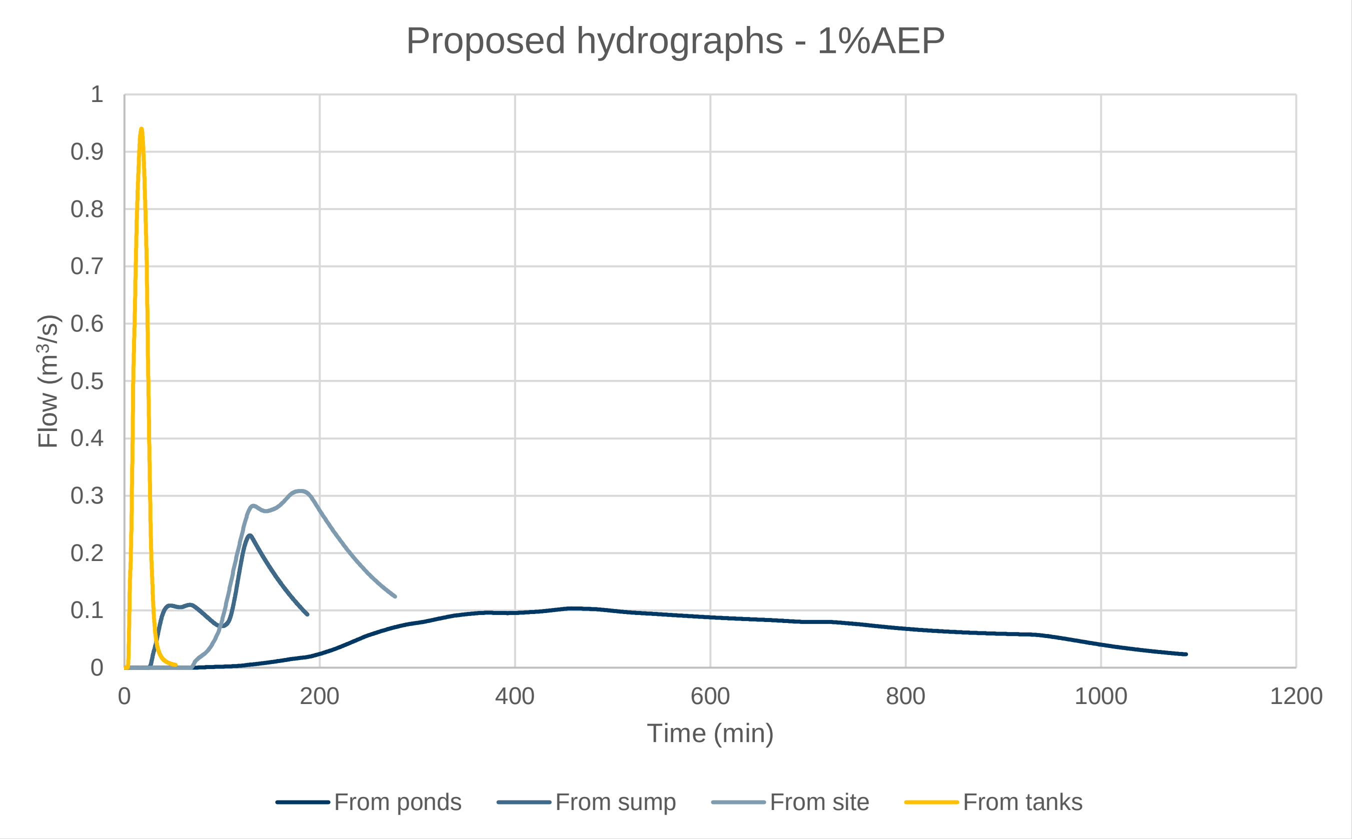


Figure 4.6 DRAINS model proposed hydrographs – 5%AEP

Results summary

A table of results is provided below in Table 4.3.

Table 4.3 DRAINS model results summary

Discharge point	Existing 20%AEP		Propsoed 20%AEP		Existing 1%AEP		Proposed 1%AEP	
	Peak flow (m³/s)	Critical duration (min)	Peak flow (m³/s)	Critical duration (min)	Peak flow (m³/s)	Critical duration (min)	Peak flow (m³/s)	Critical duration (min)
From ponds	0.10	720	0.09	360	0.14	720	0.10	720
From sump	0.24	120	0.14	120	0.38	120	0.23	120
From site	0.31	120	0.20	180	0.49	180	0.31	180
From tanks	N/A	N/A	0.73	15	N/A	N/A	0.94	15

4.3.4 Analysis of results

The proposed development to the site has an obvious impact on the existing stormwater system. Results for both the minor and major events show that discharge from the site to the existing discharge point is reduced for both the minor and major events, with the critical durations remaining the same. This is expected as the catchments from the stock yards and proposed car park and amenities building is diverted away from the sump and ponds and a new point of discharge is proposed.

Detention requirements

The proposed point of discharge from the rainwater tanks/carpark experiences relatively high flows as the model assumes the rainwater tanks are full for both the minor and major events. Council specifications and DA conditions stated that detention must be provided up to and including the 1% AEP, however, discussions with Council have confirmed that a risk based approach is to be considered for this site. This was considered a rational approach as the proposed point of discharge is directly to Blackjack Creek and the rainwater tanks are expected to be regularly used and therefore the likelihood of the tanks being full for a rainfall event is low. In the event that the rainwater tanks are full prior due to recent rainfall prior to a significant event, it is reasonable to presume that Blackjack Creek will be flowing at a higher than daily level and therefore the additional discharge from the site will not have a detrimental impact to the waterway. Figure 4.7 below provides an indicative layout for the proposed drainage from the carpark and rainwater tanks which are to discharge to Blackjack Creek.

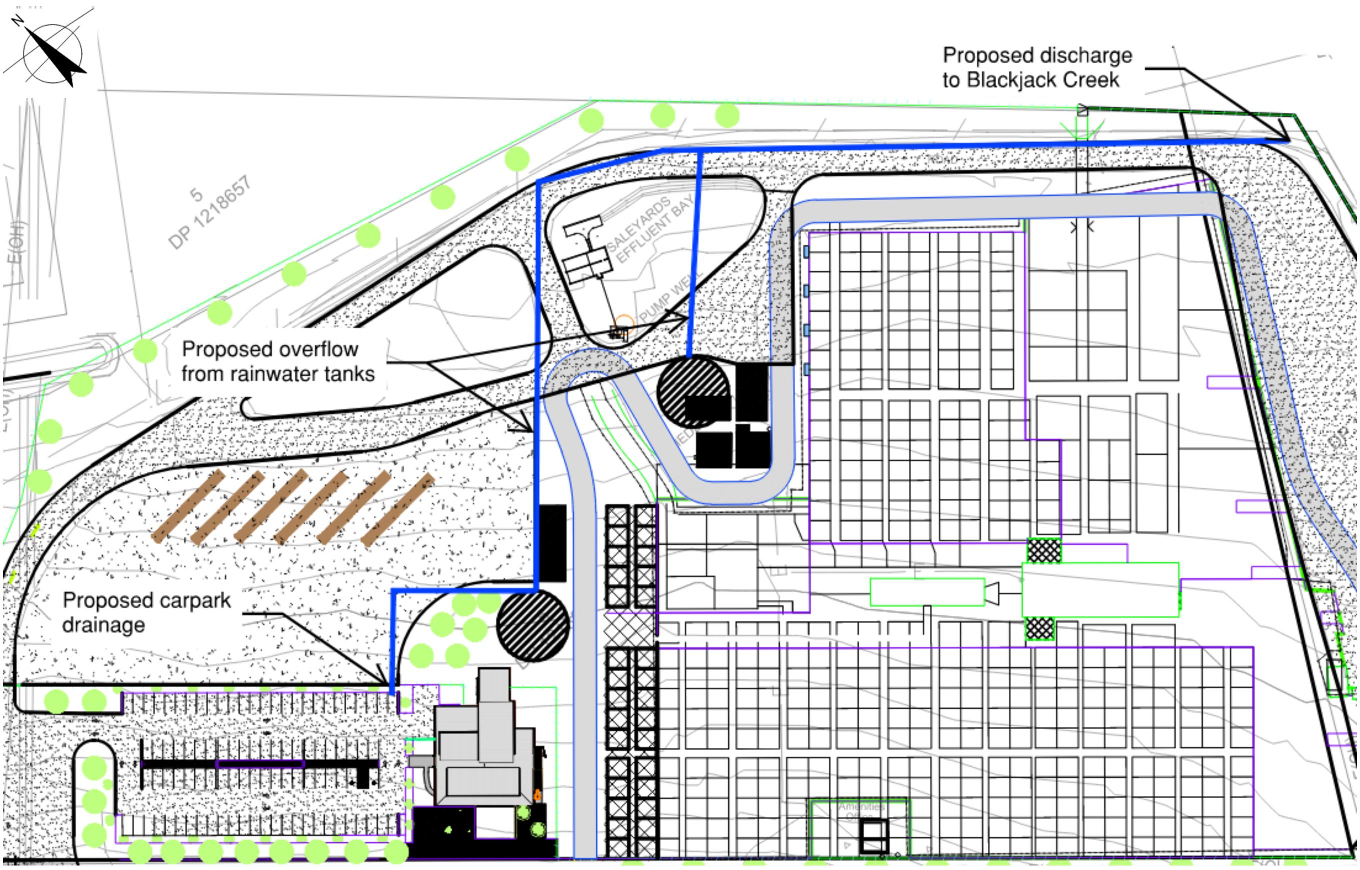


Figure 4.7 Proposed drainage layout

The existing ponds provide adequate detention for the proposed development which modifies the catchment draining to the sump and the ponds, and no additional works is required to attenuate proposed flows.

An assessment has been undertaken as part of the water balance in Section 3 which indicates the likelihood of:

– The percentage of time that various volumes of rainwater stored is equaled or exceeded. E.g. 40% of the time there will be 700 kL in the tanks assuming 1000 kL of storage and a weekly demand of 192 kL. Refer to Figure 4.8 for the assessment.
– Attenuation by comparing the inflow vs overflow. Refer to Figure 4.9 for the assessment.
– When it rains, what percentage of inflow is stored and what percentage is overflow/discharged. Refer to Figure 4.10 and Figure 4.11 for the assessment.

The assessments above were undertaken using 120 years of historical data against two different rainwater tank volumes (1000 kL and 1324 kL) and two different demands (147 kL/week and 192 kL/week).

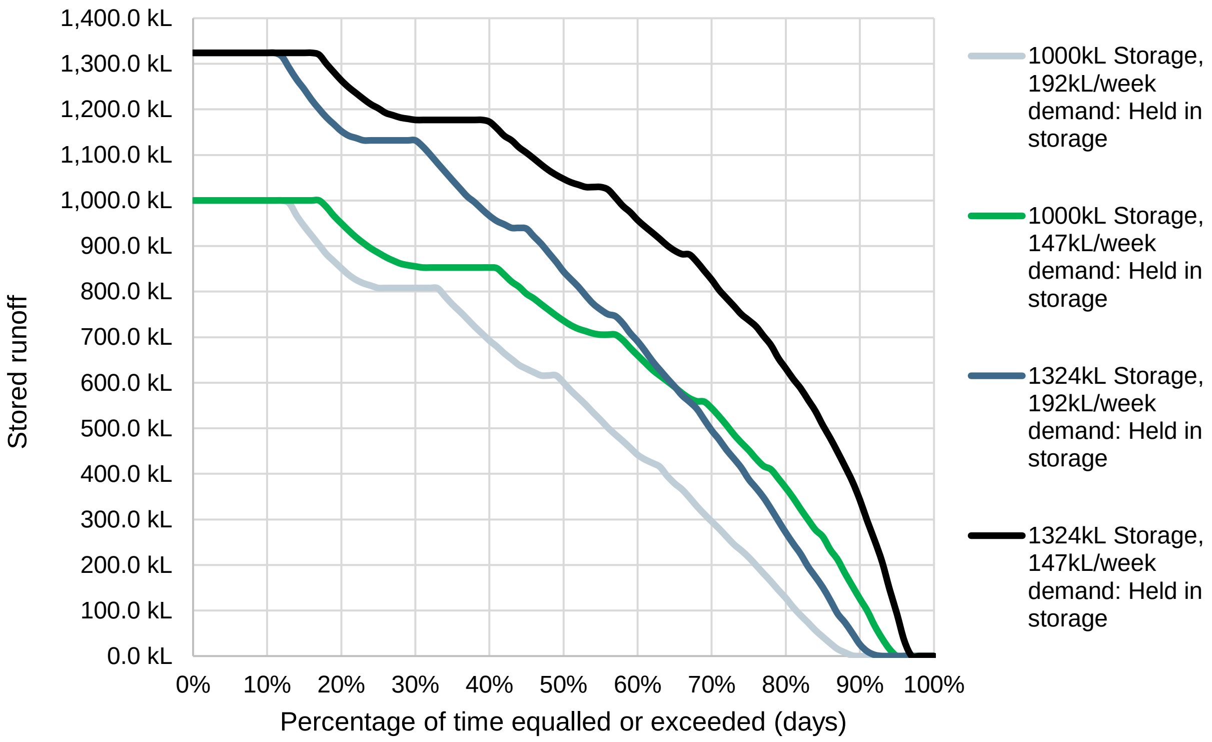


Figure 4.8 ***Percentage of time equaled or exceeded (days) for various rainwater tank sizes and demands***

Figure 4.8 above indicates that regardless of whether the tanks are 1000 kL or 1324 kL, they are full for relatively the same period of time given the same demand, which is approximately 14% for the 1000 kL tank, and 18% of the time for the 1324 kL tank. The results indicate that for the most conservative case (the lowest volume of storage with the highest demand rata), there is some water within the tank at least 86% of the time. This increases to 95% of the time for the largest tank with the lower demand.

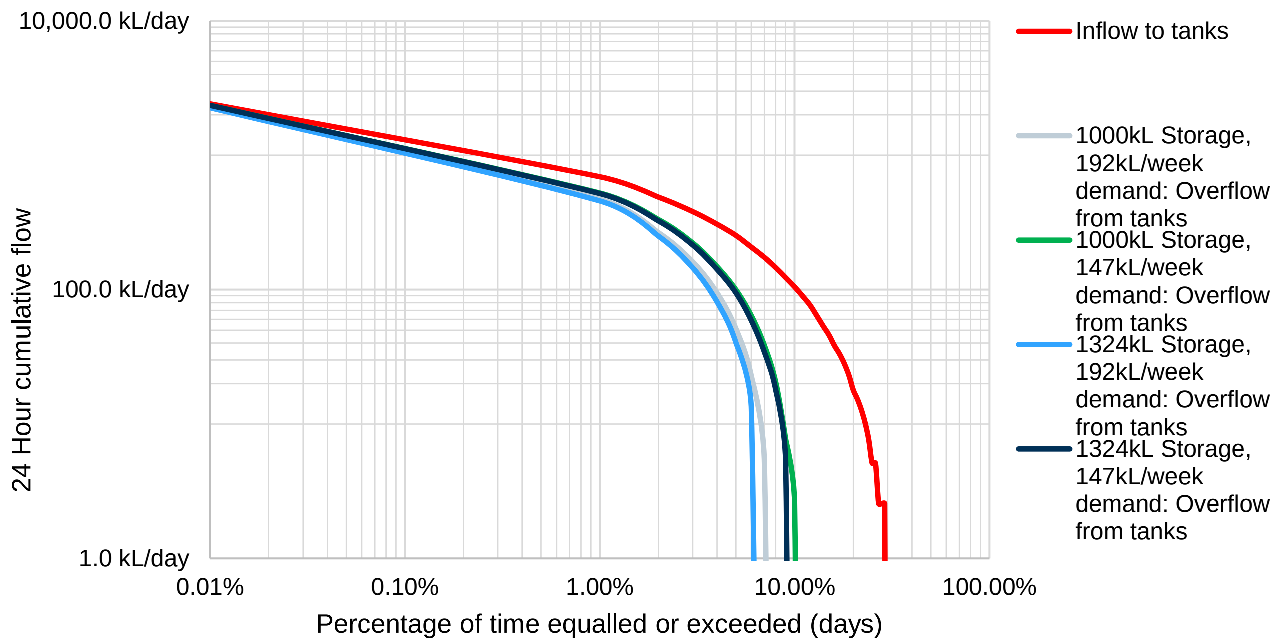


Figure 4.9 ***Attenuation provided for various rainwater tank sizes and demands***

Attenuation is determined by comparing the inflow rate to the outflow rate. Figure 4.9 demonstrates that the proposed tanks are likely to provide significant attenuation.

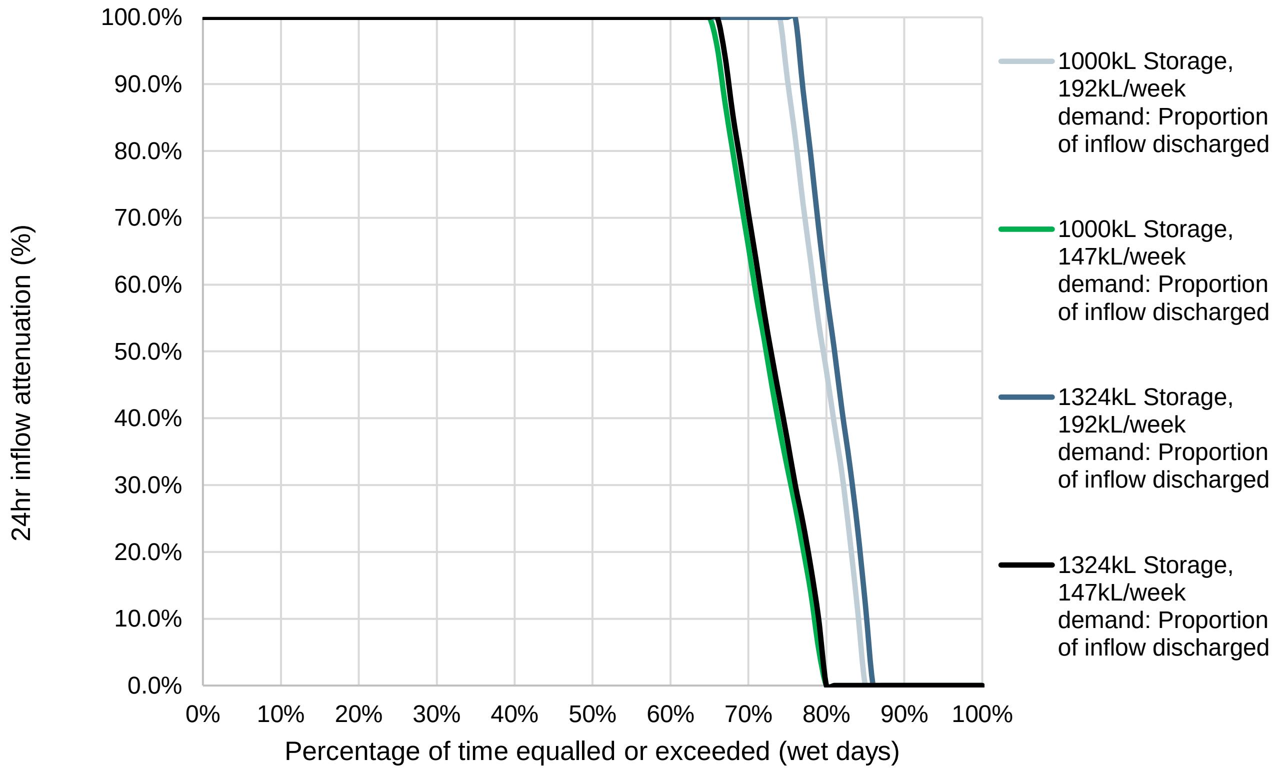


Figure 4.10 ***Percentage of attenuation provided for days where rainfall > 0 mm***

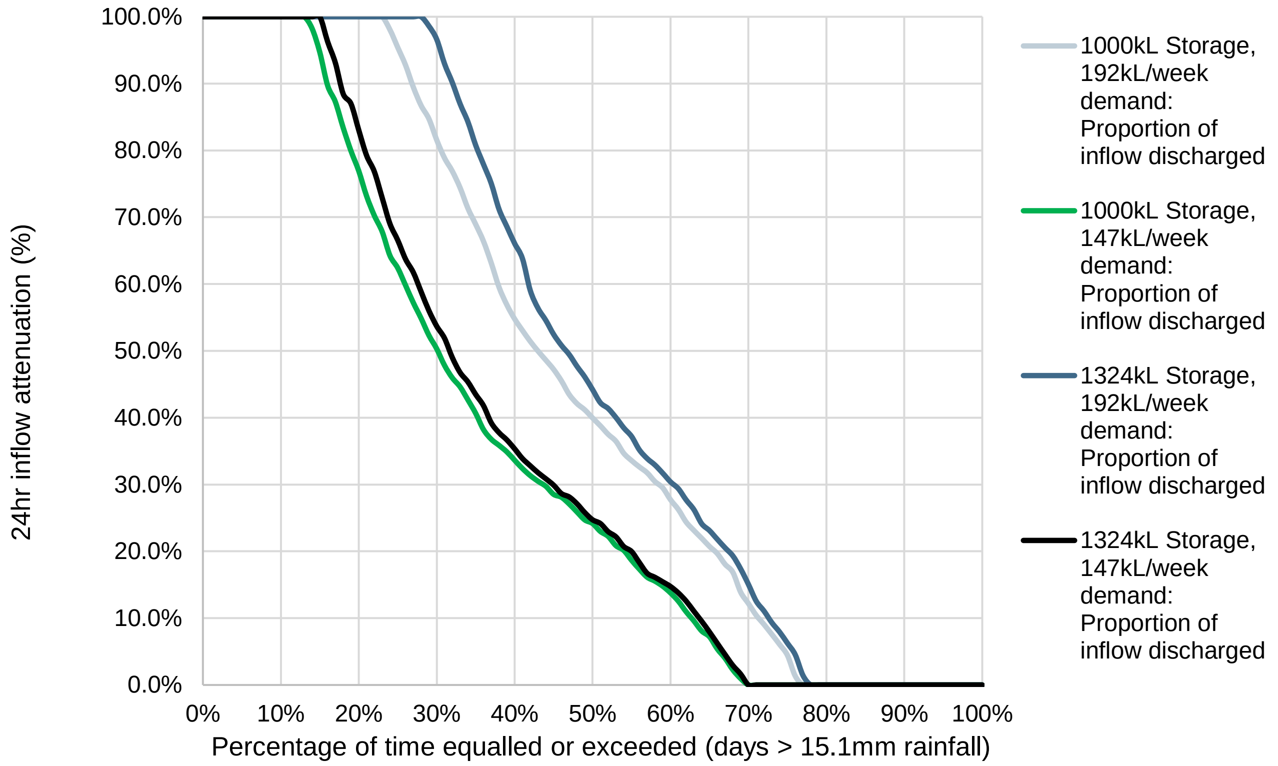


Figure 4.11 Percentage of attenuation provided for days where rainfall > 15.1 mm

Figure 4.10 and Figure 4.11 demonstrate the ratio between the daily percentage of retention vs the likelihood of exceedance. The assessment was undertaken for two scenarios: Figure 4.10 considers *any* day where rainfall was recorded. The resultant graph, while correct, can be misleading as Gunnedah generally experiences very low intensity rainfall with only a few mm of precipitation. The assessment was re-run only assessing days with rainfall recorded over 15.1 mm, which is close to the IFD rainfall depth associated with the 1%AEP, 5 minute storm of 17.5 mm, which was considered a suitable benchmark given catchment losses and uncertainty within the network. The assessment estimates that the rainwater tanks will attenuate 100% of the flow between 15 to 30% of the time, depending on the tank size and weekly usage. It also estimates that between 20 to 30% of the time there will be no attenuation provided at all.

5. Conclusion

The objective of this Stormwater Management Plan (SWMP) was to assess the impact of proposed development at the Gunnedah saleyards, and to ensure effective drainage is maintained to the lawful point of discharge for both the minor and major storm events.

5.1 Rainwater tank sizing

A water balance assessment was undertaken of potential storage volumes to meet the sites demand. Assuming the recent years demand is applicable, a minimum storage volume of 800 kL would be necessary to meet demand 90% of the time. To allow for fluctuations in demand, it may be suitable to adopt a larger storage volume.

Subject to availability and engineering requirements, a potential configuration could be adopted based upon:

– 2 x storage tanks operating as a single supply system.
– Each tank having an approximate ground footprint of 14 m diameter and approximately 4.3 m height, providing approximately 662 kL of storage.
– A total storage volume of 1,324 kL.

The results of a system adopting this size are shown below in Table 5.1:

Table 5.1 Water balance results for a 1,324 kL system

Demand	Long-term proportion of time demand is met	Long-term average weekly deficit
147.3 kL/week	95%	5.3 kL/week
192.1 kL/week	86%	18.7 kL/week
236.92 kL/week	75%	42.1 kL/week

The proposed volume of rainwater storage can be shared between multiple tanks if required. The tanks should be connected to a shared offtake if possible to allow parallel drawdown.

Based on the detention recommendations in Section 5.2 below. it is not recommended that the tanks are topped up either automatically or manually unless required (i.e. during drought periods or above-average use).

5.2 Stormwater detention

The stormwater quantity assessment has determined that the development is likely to increase peak runoff from the site, however, the risk of the intensity of runoff exceeding Council requirements is very low, given the incorporation of the recommended rainwater tanks from Section 5.1 above. As Gunnedah's climate is historically dry with low intensity rainfall, the likelihood of the proposed rainwater tanks being full prior to a significant storm is very low. In the unlikely event that the tanks are full prior to a significant event, the omission of further rainwater detention can be mitigated when discharging directly to Blackjack Creek which will likely be flowing at a higher than the average dry weather levels.

The existing ponds will adequate provide detention for the proposed modifications to the catchment, and therefore it is proposed that no additional stormwater detention is required for the development.

5.3 Detailed Design considerations

During Detailed Design, the following is recommended:

- Confirmation with Council on the volume of rainwater storage, the number, dimension and location of rainwater tanks.
- Location of the point of discharge to Blackjack Creek and the required scour protection. The scour protection should be designed for the velocities expected from both the local and regional catchments.
- Sizing of rainwater tanks and overflow pipework to contain flow up to and including the 1%AEP event. This may be reduced based on discussions with Council during Detailed Design.

Appendices

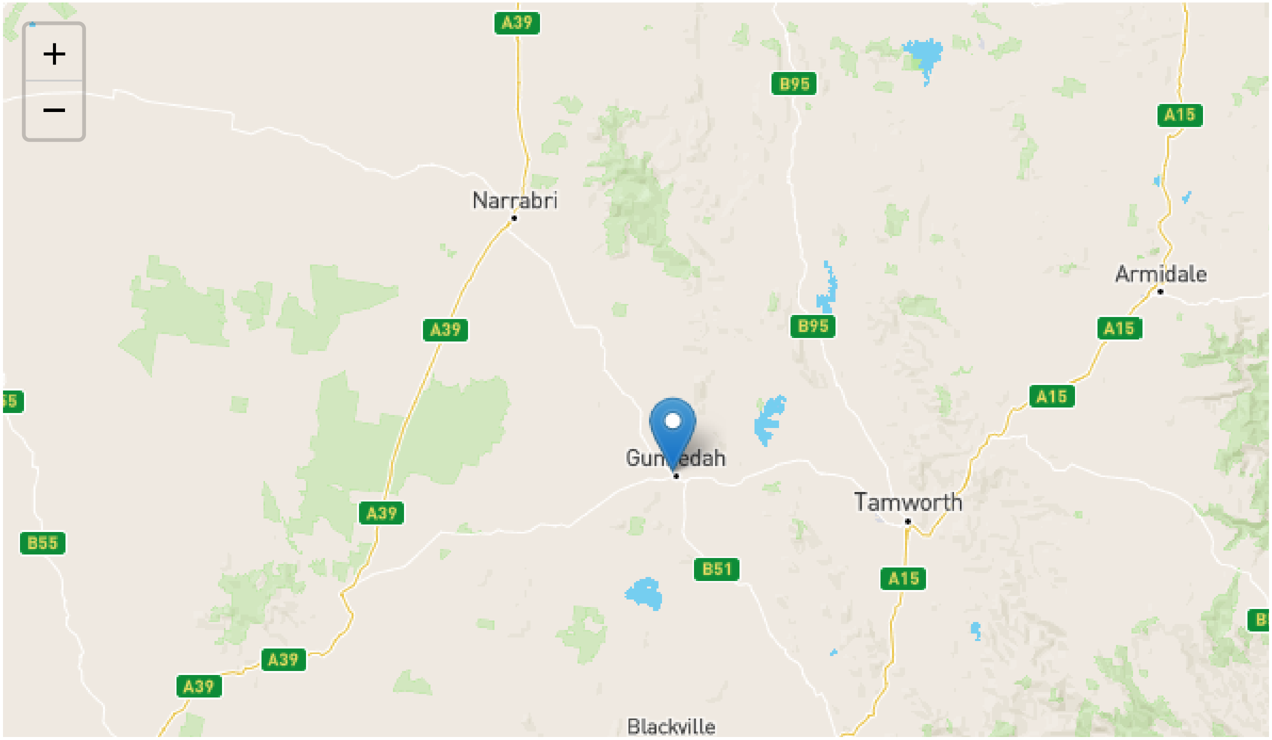
Appendix A

IFD data

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	150.24
Latitude	-30.97
Selected Regions (clear)	
River Region	show
ARF Parameters	show
Storm Losses	show
Temporal Patterns	show
Areal Temporal Patterns	show
BOM IFDs	show
Median Preburst Depths and Ratios	show
10% Preburst Depths	show
25% Preburst Depths	show
75% Preburst Depths	show
90% Preburst Depths	show
Interim Climate Change Factors	show
Probability Neutral Burst Initial Loss (./nsw_specific)	show



Data

River Region

Division	Murray-Darling Basin
River Number	20
River Name	Namoi River

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2016_v1

ARF Parameters

$$ARF = Min\left\{1, \left[1 - a\left(Area^{b} - c\log_{10}Duration\right)Duration^{-d} + eArea^{f}Duration^{g}\left(0.3 + \log_{10}AEP\right) + h10^{iArea\frac{Duration}{1440}}\left(0.3 + \log_{10}AEP\right)\right]\right\}$$

Zone	a	b	c	d	e	f	g	h	i
Semi-arid Inland QLD	0.159	0.283	0.25	0.308	7.3e-07	1.0	0.039	0.0	0.0

Short Duration ARF

$$ARF = Min\left[1, 1 - 0.287\left(Area^{0.265} - 0.439\log_{10}(Duration)\right).Duration^{-0.36} + 2.26 \text{ x } 10^{-3} \text{ x } Area^{0.226}.Duration^{0.125}\left(0.3 + \log_{10}(AEP)\right) + 0.0141 \text{ x } Area^{0.213} \text{ x } 10^{-0.021\frac{(Duration-180)^{2}}{1440}}\left(0.3 + \log_{10}(AEP)\right)\right]$$

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2016_v1

Storm Losses

Note: Burst Loss = Storm Loss - Preburst

Note: These losses are only for rural use and are **NOT FOR DIRECT USE** in urban areas

Note: As this point is in NSW the advice provided on losses and pre-burst on the NSW Specific Tab of the ARR Data Hub (./nsw_specific) is to be considered. In NSW losses are derived considering a hierarchy of approaches depending on the available loss information. The continuing storm loss information from the ARR Datahub provided below should only be used where relevant under the loss hierarchy (level 5) and where used is to be multiplied by the factor of 0.4.

ID	28428.0
Storm Initial Losses (mm)	49.0
Storm Continuing Losses (mm/h)	0.3

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2016_v1

Temporal Patterns | Download (.zip) (static/temporal_patterns/TP/CS.zip)

code	CS
Label	Central Slopes

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2016_v2

Areal Temporal Patterns | Download (.zip) (./static/temporal_patterns/Areal/Areal_CS.zip)

code	CS
arealabel	Central Slopes

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2016_v2

BOM IFDs

Click here (http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016&coordinate_type=dd&latitude=-30.97&longitude=150.24&sdmin=true&sdhr=true&sdday=true&user_label=) to obtain the IFD depths for catchment centroid from the BoM website

Layer Info

Time Accessed	09 June 2021 10:58AM

Median Preburst Depths and Ratios

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	1.2 (0.052)	1.1 (0.036)	1.1 (0.029)	1.0 (0.024)	1.1 (0.022)	1.2 (0.021)
90 (1.5)	0.4 (0.015)	0.6 (0.017)	0.8 (0.018)	0.9 (0.018)	0.9 (0.015)	0.8 (0.013)
120 (2.0)	0.8 (0.028)	1.3 (0.034)	1.7 (0.036)	2.0 (0.037)	1.3 (0.021)	0.8 (0.011)
180 (3.0)	1.2 (0.039)	1.0 (0.024)	0.9 (0.018)	0.8 (0.013)	1.7 (0.024)	2.4 (0.029)
360 (6.0)	2.1 (0.054)	2.4 (0.045)	2.6 (0.042)	2.8 (0.039)	4.0 (0.046)	4.9 (0.050)
720 (12.0)	0.1 (0.002)	0.4 (0.006)	0.6 (0.007)	0.7 (0.008)	7.5 (0.070)	12.6 (0.105)
1080 (18.0)	0.0 (0.000)	0.3 (0.004)	0.5 (0.005)	0.6 (0.006)	11.0 (0.090)	18.8 (0.137)
1440 (24.0)	0.0 (0.000)	0.1 (0.001)	0.1 (0.001)	0.2 (0.002)	7.8 (0.058)	13.5 (0.089)
2160 (36.0)	0.0 (0.000)	0.1 (0.001)	0.1 (0.001)	0.2 (0.001)	3.1 (0.020)	5.2 (0.030)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.2 (0.001)	0.3 (0.002)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

10% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
90 (1.5)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

25% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.000)	0.0 (0.001)	0.1 (0.002)	0.1 (0.002)	0.0 (0.001)	0.0 (0.000)
90 (1.5)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

75% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	11.3 (0.497)	9.6 (0.307)	8.5 (0.227)	7.4 (0.170)	9.7 (0.185)	11.4 (0.192)
90 (1.5)	9.4 (0.365)	9.0 (0.255)	8.8 (0.208)	8.5 (0.173)	8.7 (0.147)	8.8 (0.132)
120 (2.0)	11.2 (0.402)	11.9 (0.310)	12.3 (0.269)	12.7 (0.239)	13.8 (0.217)	14.7 (0.204)
180 (3.0)	13.6 (0.432)	13.0 (0.303)	12.6 (0.247)	12.3 (0.206)	16.7 (0.234)	19.9 (0.249)
360 (6.0)	15.8 (0.405)	19.3 (0.365)	21.7 (0.345)	23.9 (0.329)	23.6 (0.275)	23.4 (0.242)
720 (12.0)	8.6 (0.176)	11.4 (0.173)	13.3 (0.170)	15.1 (0.167)	34.6 (0.325)	49.2 (0.412)
1080 (18.0)	1.9 (0.034)	11.3 (0.150)	17.5 (0.197)	23.4 (0.228)	38.5 (0.316)	49.9 (0.364)
1440 (24.0)	1.2 (0.020)	5.4 (0.066)	8.2 (0.084)	10.9 (0.096)	26.1 (0.194)	37.5 (0.247)
2160 (36.0)	0.0 (0.000)	6.2 (0.067)	10.3 (0.093)	14.3 (0.111)	18.3 (0.118)	21.3 (0.121)
2880 (48.0)	0.0 (0.000)	3.1 (0.031)	5.2 (0.043)	7.2 (0.051)	7.6 (0.045)	8.0 (0.041)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	1.4 (0.007)	2.4 (0.011)

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

90% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	33.6 (1.482)	29.0 (0.928)	25.9 (0.694)	23.0 (0.528)	28.9 (0.553)	33.3 (0.563)
90 (1.5)	23.3 (0.909)	24.7 (0.701)	25.6 (0.608)	26.5 (0.539)	29.0 (0.492)	30.9 (0.463)
120 (2.0)	42.9 (1.535)	40.2 (1.048)	38.3 (0.838)	36.6 (0.686)	37.6 (0.590)	38.4 (0.533)
180 (3.0)	43.5 (1.378)	40.2 (0.935)	38.1 (0.743)	36.0 (0.604)	58.4 (0.823)	75.3 (0.939)
360 (6.0)	39.5 (1.013)	45.7 (0.864)	49.8 (0.794)	53.7 (0.740)	71.9 (0.836)	85.5 (0.884)
720 (12.0)	22.4 (0.460)	34.1 (0.517)	41.8 (0.536)	49.2 (0.547)	72.1 (0.678)	89.3 (0.747)
1080 (18.0)	19.1 (0.346)	33.7 (0.448)	43.3 (0.487)	52.5 (0.511)	72.4 (0.595)	87.3 (0.637)
1440 (24.0)	14.9 (0.248)	24.0 (0.291)	30.0 (0.307)	35.7 (0.316)	77.0 (0.572)	107.9 (0.711)
2160 (36.0)	14.4 (0.213)	29.0 (0.312)	38.6 (0.349)	47.9 (0.372)	49.3 (0.319)	50.4 (0.287)
2880 (48.0)	0.7 (0.009)	10.0 (0.099)	16.1 (0.134)	22.1 (0.157)	23.7 (0.139)	24.9 (0.128)
4320 (72.0)	0.0 (0.000)	3.5 (0.032)	5.9 (0.044)	8.1 (0.052)	17.8 (0.093)	25.1 (0.114)

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

Interim Climate Change Factors

	RCP 4.5	RCP6	**RCP 8.5**
2030	**0.972 (4.9%)**	0.847 (4.2%)	**1.052 (5.3%)**
2040	**1.225 (6.2%)**	1.127 (5.7%)	**1.495 (7.6%)**
2050	**1.452 (7.3%)**	1.406 (7.1%)	**1.971 (10.1%)**
2060	**1.653 (8.4%)**	1.685 (8.6%)	**2.480 (12.9%)**
2070	**1.827 (9.3%)**	1.963 (10.1%)	**3.023 (15.9%)**
2080	**1.974 (10.1%)**	2.241 (11.6%)	**3.599 (19.2%)**
2090	**2.095 (10.8%)**	2.518 (13.1%)	**4.208 (22.8%)**

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2019_v1
Note	ARR recommends the use of RCP4.5 and RCP 8.5 values. These have been updated to the values that can be found on the climate change in Australia website.

Probability Neutral Burst Initial Loss

min (h)\AEP(%)	50.0	20.0	10.0	5.0	2.0	1.0
60 (1.0)	22.8	22.7	18.9	19.6	19.4	18.9
90 (1.5)	25.8	25.3	21.7	22.0	21.5	21.0
120 (2.0)	28.1	21.7	19.3	19.7	20.8	19.2
180 (3.0)	31.7	22.3	20.3	20.6	19.0	15.5
360 (6.0)	38.1	22.9	20.7	20.6	19.4	15.3
720 (12.0)	42.4	29.2	26.2	25.6	18.9	11.6
1080 (18.0)	43.2	31.6	28.0	26.2	19.2	10.9
1440 (24.0)	44.8	34.2	32.4	32.2	23.8	15.2
2160 (36.0)	45.2	35.3	32.6	32.3	28.6	20.5
2880 (48.0)	48.6	40.2	39.3	39.8	37.1	30.2
4320 (72.0)	49.4	42.5	43.2	44.0	42.0	33.5

Layer Info

Time Accessed	09 June 2021 10:58AM
Version	2018_v1
Note	As this point is in NSW the advice provided on losses and pre-burst on the NSW Specific Tab of the ARR Data Hub (./nsw_specific) is to be considered. In NSW losses are derived considering a hierarchy of approaches depending on the available loss information. Probability neutral burst initial loss values for NSW are to be used in place of the standard initial loss and pre-burst as per the losses hierarchy.

Download TXT (downloads/7203eeb7-ce3c-48bd-90d7-b9b4a218a7e6.txt)

Download JSON (downloads/4022cb16-ccb2-4b9b-86e6-0b77d1aba37d.json)

Generating PDF... (downloads/a5ee5640-add7-4809-ad78-50c6539812fc.pdf)

Location

Label: Not provided

Latitude: -30.97 [Nearest grid cell: 30.9625 (S)]

Longitude:150.24 [Nearest grid cell: 150.2375 (E)]

IFD Design Rainfall Depth (mm)

Issued: 09 June 2021

Rainfall depth for Durations, Exceedance per Year (EY), and Annual Exceedance Probabilities (AEP).
[FAQ for New ARR probability terminology]

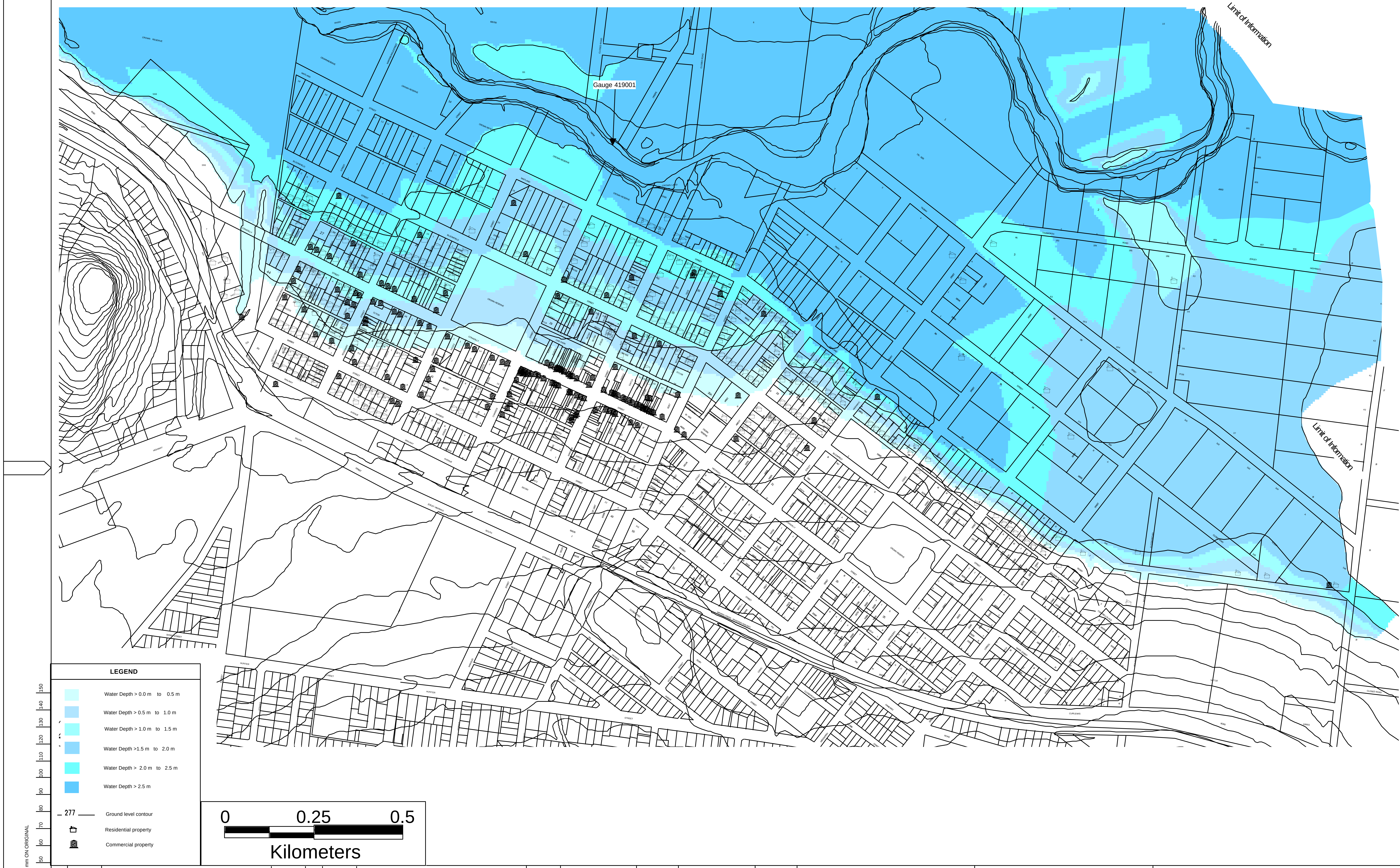
	Annual Exceedance Probability (AEP)						
Duration	**63.2%**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**
1 min	1.73	1.96	2.71	3.23	3.76	4.48	5.05
2 min	2.94	3.32	4.58	5.49	6.42	7.74	8.76
3 min	4.08	4.61	6.35	7.60	8.88	10.7	12.0
4 min	5.10	5.77	7.95	9.50	11.1	13.2	14.9
5 min	6.02	6.81	9.38	11.2	13.0	15.6	17.5
10 min	9.44	10.7	14.8	17.6	20.4	24.3	27.4
15 min	11.7	13.3	18.3	21.9	25.4	30.3	34.1
20 min	13.4	15.2	21.0	25.0	29.1	34.7	39.2
25 min	14.7	16.7	23.0	27.5	32.0	38.2	43.2
30 min	15.8	17.9	24.7	29.6	34.4	41.2	46.5
45 min	18.3	20.7	28.5	34.1	39.8	47.7	53.9
1 hour	20.1	22.7	31.3	37.4	43.6	52.3	59.2
1.5 hour	22.8	25.7	35.3	42.1	49.1	58.9	66.6
2 hour	24.8	28.0	38.3	45.7	53.3	63.7	72.0
3 hour	28.0	31.6	43.0	51.2	59.6	71.0	80.2
4.5 hour	31.7	35.7	48.5	57.6	66.7	79.3	89.3
6 hour	34.7	39.0	52.9	62.7	72.6	86.0	96.7
9 hour	39.5	44.4	60.1	71.1	82.1	97.1	109
12 hour	43.3	48.7	65.9	77.9	90.0	106	119
18 hour	49.1	55.3	75.1	88.9	103	122	137
24 hour	53.6	60.4	82.3	97.6	113	134	152
30 hour	57.0	64.4	88.1	105	122	145	164
36 hour	59.9	67.7	92.9	111	129	155	175
48 hour	64.3	72.8	101	120	141	170	194
72 hour	70.0	79.4	111	133	157	191	220
96 hour	73.5	83.6	117	141	166	204	236
120 hour	76.0	86.6	121	146	172	212	245
144 hour	77.9	88.7	124	150	175	215	249
168 hour	79.4	90.4	126	151	177	215	249

Note:
The 50% AEP IFD **does not** correspond to the 2 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 1.44 ARI.
* The 20% AEP IFD **does not** correspond to the 5 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 4.48 ARI.

This page was created at **10:59 on Wednesday 9 June 2021 (AEST)**

Appendix B

1% AEP flood map



CODE	DATE	AMENDMENT / ISSUE DESCRIPTION	APPROVAL	CODE	DATE	AMENDMENT / ISSUE DESCRIPTION	APPROVAL

TITLE	INITIAL	SIGNATURE	DATE
DRAFTSMAN			
DRAFTING CHECK			
DESIGNER			
DESIGN CHECK			
PROJECT MANAGER			

CLIENT

GUNNEDAH SHIRE COUNCIL

COUNCIL OFFICE: 63 ELGIN STREET
GUNNEDAH NSW 2380
PH (02) 6742 0422 FAX (02) 6742 0523

APPROVED DATE

SMEC
SMEC AUSTRALIA PTY LTD
ACN 065 475 149
LEVEL 6 77 PACIFIC HIGHWAY
NORTH SYDNEY NSW 2060
PH 02 9903-4411 FAX 02 9955-6113

PROJECT TITLE

GUNNEDAH FLOODPLAIN MANAGEMENT PLAN

Inundation Map - Gunnedah 1% AEP

SCALE	DRAWING STATUS	PROJECT / DRAWING No.	ISSUE
1 : 7500	PRELIMINARY	31923-003	01