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Drainage and Sewer Report

MJM
CONSULTING ENGINEERS
CIVIL • STRUCTURAL • BUILDING DESIGN • PLANNING

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68 Gregadoo Road – Lake Albert, Wagga Wagga 2650

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1. Introduction

The project is intended to subdivide approximately 78ha parcel of land, highlighted in Figure 1, by providing roads, drainage, sewer and associated services for the creation of 141 residential lots as observed in Figure 2. The existing area is located in the suburb of Lake Albert, Wagga Wagga. The drainage and sewer design strategy used is intended to maintain the objectives for land subdivisions within the Wagga Wagga city council as follows:

- provide safety for the public;
- minimise and control, nuisance flooding and to provide for the safe passage of less frequent flood;
- protect property;
- enhance the urban landscape;
- maximise the land area available for dwellings;
- minimise the environmental impact of urban runoff;
- ensure discharge rates from new developments, do not exceed the capacity of the downstream stormwater systems nor result in additional scour and instability of natural creek, river systems and artificial channels;
- conform to natural drainage patterns and discharge to natural drainage paths in the catchment.

Figure 1. Catchment Area

2. Drainage System

2.1. Minor Drainage System

The minor drainage system includes underground drainage, junction pits, access chambers and outlet structures designed to fully contain and convey the discharge stormwater from the minor storm (QUDM, 2013). This arrangement also include:

• Inter-allotment drainage pits installed to collect surface runoff from within allotments, as well as the roof-water drainage provisions for buildings.

A 10% Annual Exceedance Probability (AEP) was adopted in the design of the minor system as per Wagga Wagga City Council (WWCC) guidelines.

2.2. Major Drainage System

The major drainage system is that part of the overall drainage system designed to convey a specified major storm flow (QUDM, 2013). This system comprises:

• Floodway channels & road reserves designed to carry flows in excess of the capacity of the minor drainage system

• Constructed waterways and swales.

A 1% AEP was adopted in the design of the major system as per WWCC guidelines.

Figure 2. Proposed Subdivision

2.3. Stormwater catchment areas

Based on topographic information of the project plan, three catchment areas are defined and depicted in Figure 3. Catchment area 1 shown by green colour has an area 426295 m^2, followed by catchment areas 2 (red) and 3 (purple) covering 227400 m^2, 125151 m^2 of the land, respectively. In addition, overland flow paths for each of catchments areas 1, 2, 3 are shown in the figure which end up to points A, B, C, as depicted in the figure. The length and land slope of the flow paths are presented in Table 1. The calculation of peak flood rate as well as runoff travel time will be calculated for each of these catchment areas separately for two cases of existing and developed land.

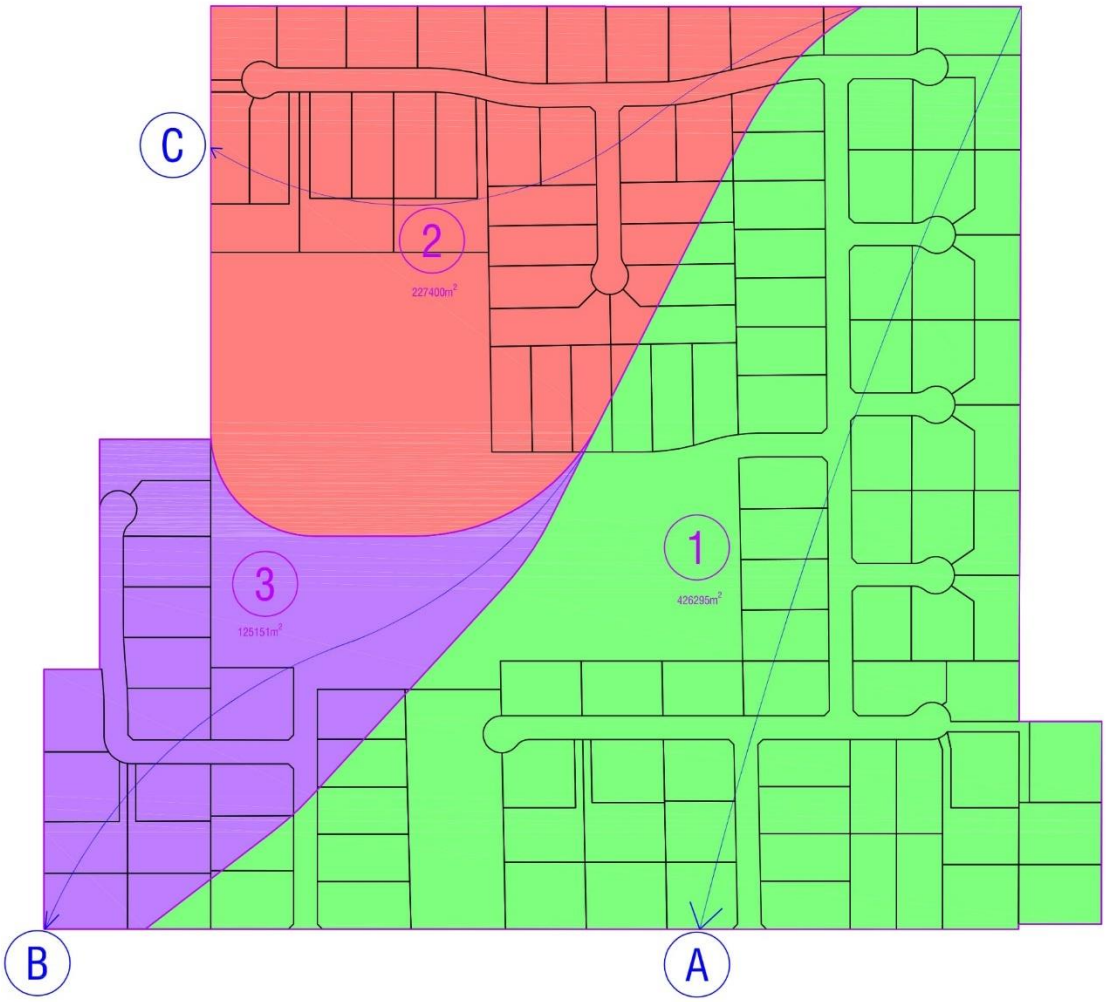


Figure 3. Catchment areas and overland flow paths

Table 1. flow paths

Catchment area	Area (ha)	Length (m)	Slope (%)
A	42.6295	950	1.6
B	22.7400	700	1.3
C	12.5151	750	1.1

2.4. Estimation of Peak Flood

The maximum or peak flood value generated by rainfall is required for the design of many hydraulic structures such as storage and detention reservoirs, highway and railway culverts, urban sewers lines, and stormwater network. To estimate the magnitude of a flood peak when direct measurements cannot be obtained, the following methods are used:

- Rational method
- Empirical methods
- Unit hydrograph methods

The use of a particular method depends upon the desired objective, the available data and the importance of the project (Rakhecha & Singh, 2009).

2.4.1. Estimation of flowrates by the rational method

This method of estimating the peak flood discharge is based on physical and hydraulic properties of catchments and their effects on storm rainfall. The peak flood discharge value is given as:

$$Q = \frac{CIA}{360} \qquad \text{Equation 1}$$

where Q is peak flood discharge in $^{m^3}/_{s}$, C is runoff coefficient, A is catchment area in ha, and I is rainfall intensity in $^{mm}/_{hr}$ with the selected recurrence interval in years and duration equal to the catchment's time of concentration in minutes. The time of concentration is defined as the time which would be required for surface runoff from the remotest part of the catchment to reach the outlet or the point of interest on the water course. The time varies depending on the slope and characteristics of land surfaces. There are a number of empirical equations for the estimation of the time of concentration. Coefficient C represents the integrated effect of catchment losses and therefore depends upon the nature of the surface, surface slope, and rainfall intensity. The rainfall intensity corresponding to the rainfall duration and the desired recurrence interval for the catchment can be found from rainfall Intensity-Frequency-Duration (IFD) curves.

2.5. Surface Runoff and Travel Times

Overland flow before discharge to stormwater pipe system should be considered in stormwater design. The recommended approach by Australian Rainfall and Runoff (AR&R) and WWCC (WWCC, 2017) to determine time of concentration for overland flow is Kinematic Wave equation shown below.

$$T_1 = \frac{6.94\,(Ln^*)^{0.6}}{{i_1}^{0.4}\,S^{0.8}} \qquad \text{Equation 2}$$

Inputs in Kinematic Wave equation are flow path length (L), land slope (S), and surface roughness (n*), whereas overland flow time (t_1) and rainfall intensity (i_1) are outputs.

There is a limitation in application of Kinematic Wave equation which restricts the maximum length of flow path to 100 meters. Since the flow path length in this project is more than the maximum value allowed in the formula, the overall time of concentration is calculated in two stages. In first stage, Kinematic Wave is utilized for the first 100m length of the flow path. Then in second stage, manning's equation for open channel flows are utilized to find out time of concentration in second part of flow path. Application of manning's equation is justifiable in this case because it is assumed that a naturally overland flow is generated into an earth lined open channel which flows through the lowest elevation along the flow path in each catchment area. Manning's equation is presented below.

$$Q_2 = \frac{AR^{\frac{2}{3}}S^{0.5}}{n} \qquad \text{Equation 3}$$

Where A is cross sectional area of overland flow, R is hydraulic radius, S is longitudinal land slope, and n is manning's roughness coefficient. After realization of manning's flow, section details, and path length, time of concentration can be calculated.

2.5.1. Kinematic Wave equation

In this section, runoff travel time for the first 100m of overland flow is calculated. Length of flow is clearly considered to be 100m. Surface roughness (n*) is assumed to be $n^* = 0.07$ as suggested by Table 4.8.2 (WWCC, 2017) for surface type of bare clay and eroded loam soil. According to Figure 2, topographic information indicates that a decrease in elevation of 220m to 206m occurs in the length of overland flow (i.e. 950m) in catchment area A. Accordingly, land slope (S) is calculated to be %1.5. The slope is calculated as %1.1 and %0.95 for catchment areas B and C in the same way. Table 2 presents the results of Kinematic Wave approach.

Table 2. Kinematic Wave equation

Catchment area	Length (m)	Roughness, n*	Slope (%)	ARI (years)	Intensity (i_1)	Time of Concentration, T_1 (min)
A	100	0.07	1.5	10	71.058	14.787
	100	0.07	1.5	100	130.464	11.705
B	100	0.07	1.15	10	67.801	16.266
	100	0.07	1.15	100	124.705	12.855
C	100	0.07	0.95	10	65.487	17.429
	100	0.07	0.95	100	120.639	13.759

2.5.2. Rational method

According to rational method, three parameters are needed to determine the flow; catchment area, runoff coefficient, and rainfall intensity.

Ratio of impervious area is required to figure out the runoff coefficient. All impervious area for the existing land condition are measured through six maps and shown in Table 3. However, since the future land development is not taken place yet and impervious area for developed land

condition cannot be directly measured, an assumption is made. A portion of the land which is already developed is considered (Figure 4) and its impervious ratio is assumed to be approximately the same for the whole catchment area after development. As it can be seen in this figure, all impervious areas including buildings, pools, parking lots, and roadways are measured and shown. Since the total area of the sample is known (i.e. 3.927 ha), the ratio of impervious is simply calculated as 0.186. This impervious ratio is assumed to be approximately the same for the catchment areas A, B, and C. The impervious ratio is then used to find out run off coefficient (C) which is needed in calculation of natural catchment discharge flow (Q). Based on Table 4.13.1 in part 3 of the WWCC engineering guidelines for subdivisions and development standards (WWCC, 2017), runoff coefficient is interpolated and shown in Table 3 and Table 4. In addition, the frequency factor for runoff coefficient is determined for annual recurrence intervals of 10 and 100 years.

Figure 4. A sample of developed land

Table 3. Natural catchment flow calculations for existing land

Catchment area	**A**		**B**		**C**	
ARI, years	10	100	10	100	10	100
Total area, ha	42.6295		22.74		12.5151	
Impervious area (A), ha	3.41		1.15		1.30	
Impervious ratio	0.080		0.051		0.104	
Runoff coefficient (C)	0.246	0.292	0.225	0.271	0.263	0.314
Frequency factor	1	1.2	1	1.2	1	1.2
Frequency factor coefficient	0.246	0.350	0.225	0.325	0.236	0.376

Table 4. Natural catchment flow calculations for developed land

Catchment area	**A**		**B**		**C**	
ARI, years	10	100	10	100	10	100
Total area, ha	42.6295		22.74		12.5151	
Impervious area (A), ha	8.03		4.12		2.38	
Impervious ratio	0.188		0.181		0.190	
Runoff coefficient (C)	0.322	0.389	0.317	0.383	0.323	0.391
Frequency factor	1	1.2	1	1.2	1	1.2
Frequency factor coefficient	0.322	0.467	0.317	0.383	0.323	0.470

2.5.3. Manning's equation

In this section, runoff travel time for the second part of overland flow is calculated. It is assumed that a naturally overland flow is concentrated and flown through an earth trapezoidal-shape open channel with base width of five meters and bather slopes of 1 in 10 either side. Manning's roughness coefficient (n) is assumed to be $n = 0.035$ according to Table 4.8.3 (WWCC, 2017) for surface type of earth with weeds or gravel. Land slope (S) is considered as calculated in Kinematic Wave section for each catchment areas.

Manning's flow (Q_2) can be calculated for any arbitrary cross sectional area of flow. After the manning's flow (Q_2) is determined, velocity of flow can be simply calculated having the cross sectional area of overland flow (A). Then, time of concentration (T_2) for second part of overland flow path can be calculated using the flow velocity and path length. Eventually, critical time which by definition is the overall time of concentration required for overland runoff to reach the outlet from the remotest part of the catchment can be calculated as the summation of time of concentrations calculated for Kinematic Wave (T_1) and manning's equations (T_2).

However, since the cross sectional area of overland flow (A) and consequently the depth of water in the open channel is unknow, trial and error approach is used as the problem solving method to determine overland flow based on trial water depth values. The process starts with calculating manning's flow (Q_2) and manning's time of concentration with a trail depth of water. Manning's time of concentration is added to the Kinematic Wave one to work out the critical time from which intensity of flow can be measured based on Bureau of Meteorology (BOM) data (BOM, 2016) shown in Figure 6. This intensity is then substituted into Equation 1 to calculate rational method

flow (Q) for the whole catchment. Trial continues until manning's flow (Q_2) is very close or equal to the rational method flow (Q) for the whole catchment area. The reason is that the flow of overland stormwater is expected to be constant throughout the whole path length as a steady flow. Results of a complete trial and error method for catchment A in developed land condition are presented in Table 5 for ARI 10 and 100 years. The same method is applied to catchment areas B and C and summary of the results is shown in Table 6.

Table 5. Manning's calculations of catchment A for ARI 10 & 100 years

Depth of water	Wet area (A)	Manning's flow (Q_2), m^3/s	Velocity	Time of Concentration, T_2 (min)	Critical Time, (min)	Intensity from BOM	Flow of total path, Q	Q_2=Q?
0.15	0.98	0.84	0.86	16.50	31.29	53.5	2.04	No
0.20	1.40	1.41	1.01	14.00	28.79	56.43	2.15	No
0.25	1.88	2.15	1.14	12.40	27.19	58.47	2.23	No
0.256	1.94	2.24	1.16	12.20	26.99	58.74	2.24	Yes (ARI 10yr)
0.30	2.40	3.04	1.27	11.20	25.99	60.13	2.29	No
0.35	2.98	4.10	1.38	10.30	25.09	61.45	2.34	No
0.40	3.60	5.34	1.48	9.50	21.21	107.51	5.95	No
0.424	3.92	6.00	1.53	9.2	20.91	108.41	6.00	No
0.45	4.275	6.77	1.58	8.9	20.61	109.33	6.05	Yes (ARI 100yr)
0.5	5	8.39	1.68	8.4	20.11	110.90	6.14	No
0.55	5.775	10.22	1.77	8	19.71	112.20	6.21	No

Table 6. Summary of times of concentration for catchment areas

Pre/post construction	Catchment area	ARI	Depth of water	Rational or Manning's flow (Q= Q_2), m^3/s	Time of Concentration, T_2 (min)	Time of Concentration, T_1 (min)	Critical Time, T (min)
Existing land	A	10	0.219	1.67	13.3	14.8	28.1
		100	0.362	4.38	10.1	11.7	21.8
	B	10	0.157	0.8	13	16.3	29.3
		100	0.267	2.13	9.6	12.9	22.5
	C	10	0.12	0.45	18	17.4	35.4
		100	0.209	1.22	13.1	13.8	26.9
Developed land	A	10	0.256	2.24	12.2	14.8	27
		100	0.45	6.77	8.9	11.7	20.6
	B	10	0.192	1.15	11.6	16.3	27.9
		100	0.325	3.10	8.6	12.9	21.5
	C	10	0.137	0.57	16.7	17.4	34.1
		100	0.238	1.55	12.2	13.8	26

2.6. Existing and developed land conditions comparison

The existing and developed land conditions are compared in terms of peak flow rate and time of concentration. The results are shown in Table 7. Peak flow rate is higher for the developed case whereas critical time is lower. The reason is that impervious ratio increases as a result of construction and land development which leads to higher runoff coefficient and consequently higher flow rate (recall rational method). On the other hand, this higher flow rate means higher velocity of flow in manning's equation and less time of concentration needed for the overland rainfall to flow through the same path distance.

Table 7. Existing and developed land comparison

Pre/post construction	Catchment area	ARI	Overland flow, m^3/s	Increase in flow, %	Critical Time, (min)	Decrease in critical time, %
Existing land	A	10	1.67	-	28.1	-
		100	4.38	-	21.8	-
	B	10	0.8	-	29.3	-
		100	2.13	-	22.5	-
	C	10	0.45	-	35.4	-
		100	1.22	-	26.9	-
Developed land	A	10	2.24	34.1	27	-3.9
		100	6.77	54.6	20.6	-5.5
	B	10	1.15	43.8	27.9	-4.8
		100	3.10	45.5	21.5	-4.4
	C	10	0.57	26.7	34.1	-3.7
		100	1.55	27	26	-3.3

3. Sewer System

Sewer Lines and catchment points are depicted in Figure 5. There are one existing (E) and two other sewer catchment outlets (D, F) which will be built in this project as shown in the figure. The existing sewer area (No. 2) which connect to outlet E is hatched with oblique green lines, while the catchment areas discharging to sewer outlets D and F (No. 1, 3) are hatched with vertical red and horizontal magenta lines, respectively. The number of tenements within each sewer catchment area is presented in Table 8. Furthermore, pipe size and minimum grade for each sewer outlet are determined based on WWCC engineering guideline (WWCC, 2017) and shown in Table 8.

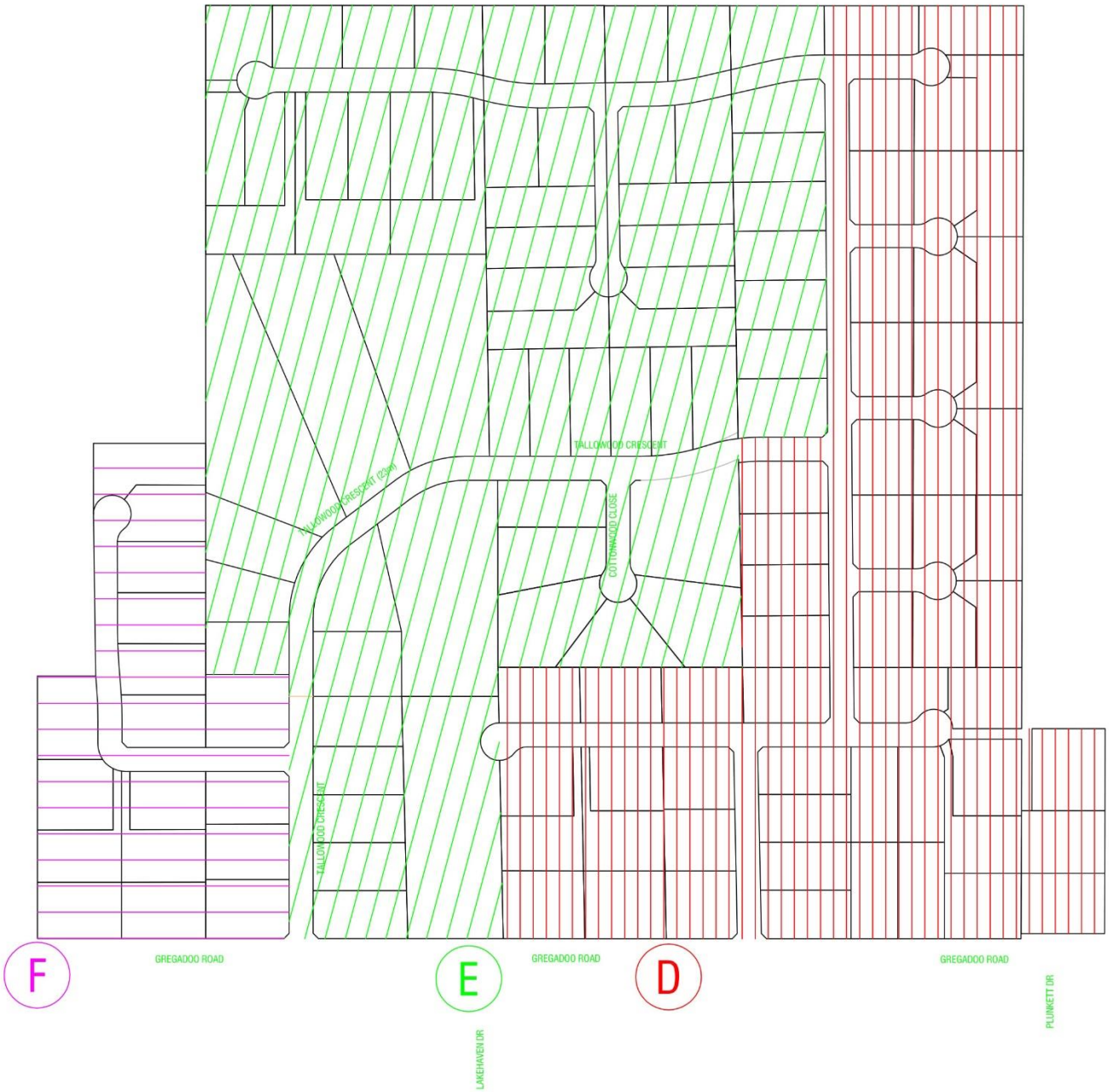


Figure 5. Sewer Lines and catchment points

Table 8. Sewer catchment area

Sewer catchment area	Sewer catchment outlet	Existing line	Number of tenements	Pipe size, mm	Min grade
1	D	No	55	150	1 in 150 (%0.67)
2	E	Existing	62	150	1 in 150 (%0.67)
3	F	No	17	150	1 in 137 (%0.73)

The minimum grade of sewer outlets D and E were interpolated as 1 in 199 (%0.5) and 1 in 207 (%0.48) according to the corresponding appendix table of WWCC engineering guidelines, however, the grade 1 in 150 (%0.67) is considered for these two outlets since this is the minimum grade value allowed by the council. In terms of pipe size, minimum sewer pipe main diameter is 150mm according to the abovementioned guideline which can satisfy requirements of this project and therefore is selected for all sewer outlets.

4. References

NSW Spatial Services. (n.d.). SIX Maps. Retrieved from SIX Maps: https://maps.six.nsw.gov.au/

QUDM. (2013). Queensland Urban Drainage Manual - Third edition. Department of Energy and Water Supply.

Rakhecha, P., & Singh, V. P. (2009). Applied hydrometeorology: Springer Science & Business Media.

WWCC. (2017). Engineering guidelines for subdivisions and development standards.

BOM. (2016). Design Rainfall Data System Retrieved from http://www.bom.gov.au/water/designRainfalls/revised-ifd/

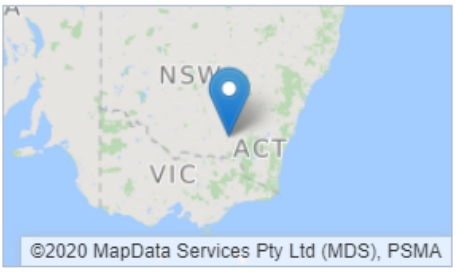
5. Appendix

Location

Label: Wagga Wagga

Latitude: -35.1841 [Nearest grid cell: 35.1875 (S)]

Longitude: 147.3678 [Nearest grid cell: 147.3625 (E)]



IFD Design Rainfall Intensity (mm/h)

Issued: 27 February 2020

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

Table | Chart

Unit: mm/h

	Annual Exceedance Probability (AEP)						
Duration	**63.2%**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**
1 min	104	118	166	200	233	278	314
2 min	88.5	101	141	169	196	234	263
3 min	80.3	91.7	128	153	178	213	239
4 min	74.0	84.5	118	142	165	197	222
5 min	68.9	78.7	110	132	154	184	207
10 min	52.2	59.7	83.8	101	118	141	159
15 min	42.6	48.8	68.6	82.4	96.3	115	130
20 min	36.3	41.6	58.5	70.3	82.1	98.2	111
21 min	35.3	40.4	56.8	68.3	79.8	95.5	108
22 min	34.4	39.3	55.3	66.5	77.7	92.9	105
23 min	33.5	38.3	53.9	64.7	75.7	90.5	102
24 min	32.7	37.4	52.5	63.1	73.7	88.2	99.5
25 min	31.9	36.5	51.2	61.6	71.9	86.0	97.1
26 min	31.1	35.6	50.0	60.1	70.2	83.9	94.7
27 min	30.4	34.8	48.9	58.7	68.6	82.0	92.5
28 min	29.7	34.0	47.8	57.4	67.0	80.1	90.4
29 min	29.1	33.3	46.7	56.1	65.6	78.4	88.4
30 min	28.5	32.6	45.7	55.0	64.2	76.7	86.6
31 min	27.9	31.9	44.8	53.8	62.8	75.1	84.7
32 min	27.3	31.3	43.9	52.7	61.6	73.6	83.0
33 min	26.8	30.7	43.0	51.7	60.4	72.1	81.4
34 min	26.3	30.1	42.2	50.7	59.2	70.7	79.8
35 min	25.8	29.5	41.4	49.8	58.1	69.4	78.3
36 min	25.4	29.0	40.7	48.8	57.0	68.1	76.8
37 min	24.9	28.5	40.0	48.0	56.0	66.9	75.4
38 min	24.5	28.0	39.3	47.1	55.0	65.7	74.1

Figure 6. Design rainfall intensity