8. DESIGN EVENT MODELLING

8.1. Overview

ARR2019 guidelines for design flood modelling were adopted for this study, including the use of ARR2019 design information for the 50%, 20%, 10%, 5%, 1%, 0.5% and 0.2% AEP events. The PMF event was derived using the Bureau of Meteorology's Generalised Short Duration Method (GSDM) (Reference 13) to estimate the probable maximum precipitation (PMP).

The flows generated by the WBNM model for each design flood event were used as inflows in the calibrated TUFLOW model to define the flood behaviour across the catchment. The ARR2019 temporal patterns, the procedure for the selection of the critical pattern duration and adopted hydrologic model parameters are discussed in the following sections. The resulting flood behaviour simulated in the TUFLOW model is subsequently presented, including an analysis of the results.

8.2. ARR2019 IFD

ARR2019 IFD rainfall information was obtained from the Bureau of Meteorology (BoM). IFD information was sourced for each sub-catchment individually from the BoM's gridded IFD data and applied in the WBNM hydrologic model. A summary of average design rainfall depths across the catchment is provided in Table 11.

Table 11: Average Design Rainfall Depths (mm)

Duration	AEP						
(min)	50%	20%	10%	5%	2%	1%	0.5%	0.2%
30	26	35	41	47	55	61	67	76
45	30	40	47	54	63	70	76	87
60	33	44	51	59	69	77	83	95
90	38	50	58	66	78	87	94	107
120	42	54	63	72	85	95	103	117
180	48	62	72	83	97	109	118	133
270	55	71	83	96	113	127	137	155
360	61	80	93	108	127	143	154	174
540	71	94	111	128	153	173	185	209
720	80	107	126	147	176	198	213	240
1080	94	127	152	178	213	242	261	295

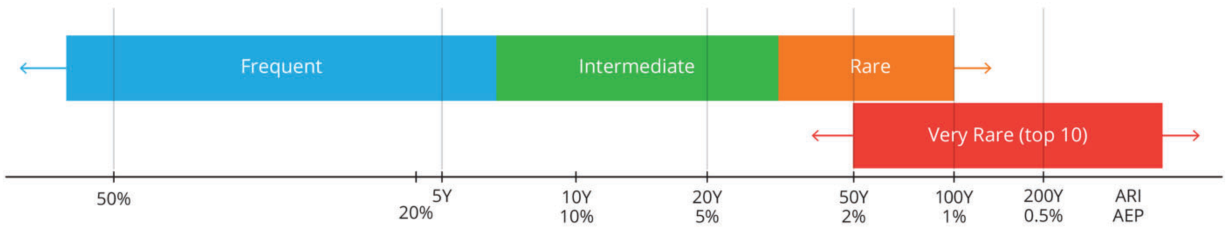
8.3. Temporal Patterns

Temporal patterns are a hydrologic tool that describe how rainfall falls over time and are often used in hydrograph estimation. Previously in ARR1987, a single burst temporal pattern has been adopted for each rainfall event duration. However ARR2019 (Reference 1) discusses the potential inaccuracies with adopting a single temporal pattern, and recommends an approach where an ensemble of different temporal patterns are investigated.

Temporal patterns for this study were obtained from the ARR2019 data hub (Reference 1, http://data.arr-software.org/). A summary of the data hub information at the catchment centroid is presented in Appendix A. There are a wide variety of temporal patterns possible for rainfall events of similar magnitude. This variation in temporal pattern can result in significant effects on the estimated peak flow. As such, the recommended methodology is to consider an ensemble of design rainfall events and determine the median catchment response from this ensemble.

The ARR2019 method divides Australia into 12 temporal pattern regions, with the Greendale Creek catchment falling within the South East Coast (NSW) region. ARR2019 provides 30 patterns for each duration, which are sub-divided into three temporal pattern bins based on the frequency of the events. Diagram 2 shows the three categories of bins (frequent, intermediate and rare) and corresponding AEP groups. The “very rare” bin is in the experimental stage and was not used in this flood study. There are ten temporal patterns for each AEP/duration in ARR2019 that were utilised in this study for the 50% AEP to 0.2% AEP events.

Diagram 2: Temporal Pattern Bins



The method employed to estimate the PMP utilises a single temporal pattern (Reference 13).

8.4. Critical Duration

The critical duration is the temporal pattern and duration that best represents the flood behaviour (e.g. flow, level) for a specific design magnitude. It is generally related to the catchment size, as flow takes longer to concentrate at the outlet from a larger catchment, as well as other considerations like land use, shape, stream characteristics, etc.

With ARR2019 methodology, the critical duration is the storm duration that produces the highest mean flow or level at a point of interest (where the mean is calculated from the ensemble of ten temporal patterns for that duration). Where there are multiple locations of interest with different contributing catchment sizes, there can be multiple critical durations that need to be considered.

Once the critical duration is established, it is usually desirable to select a representative design storm temporal pattern that reproduces this behaviour for all points of interest. This representative storm can then be used for determining design flood behaviour and for future modelling to inform floodplain management decisions.

For this study, there are two primary flood mechanisms of interest:

1. Mainstream / Lagoon: The dominant flood mechanism in the lower catchment is

mainstream flooding from Greendale Creek and Curl Curl Lagoon; and

2. Overland: The small creeks and urban drainage lines within the upper catchment which have smaller contributing catchment areas and fast runoff processes due to urbanisation.

A range of storm durations from 15 minutes to 360 minutes were run through the TUFLOW hydraulic model. For each AEP, a single representative storm was able to be selected that produced peak flood levels closest to (and slightly above) the mean ensemble peak flood levels at every point in the catchment. The adopted critical duration and representative temporal pattern for each event is shown in Table 12.

Table 12: Design Event Critical Durations and Representative Temporal Patterns

Design Event	Critical Duration	Temporal Pattern ID
50% AEP	45 minutes	TP4551
20% AEP	60 minutes	TP4565
10% AEP	60 minutes	TP4565
5% AEP	60 minutes	TP4565
2% AEP	45 minutes	TP4496
1% AEP	45 minutes	TP4496
0.5% AEP	45 minutes	TP4496
0.2% AEP	45 minutes	TP4496
PMF	30/60/120 minutes	GSDM

For the PMP the full range of applicable GSDM durations from 15 minutes to 6 hours was run through the hydraulic model to determine the critical duration for the study area.

For the PMF it was found that the 120 minute storm produced the peak flood levels within Curl Curl Lagoon, the 60 minute was within Greendale Creek and the Brookvale industrial area, and the 30 minute duration was critical in urban overland flow areas. These durations were run and the maximum taken at each location ("enveloped") to produce the flood mapping.

8.5. Design Rainfall Losses and Pre-Burst Rainfall

NSW State Government guidance for ARR2019 implementation (Reference 14) was followed to select appropriate losses for use in design flood modelling. Design rainfall losses were obtained from the ARR2019 Datahub (http://data.arr-software.org/).

Probability neutral burst initial losses were applied directly to the design storm bursts modelled. The continuing loss value from the Datahub was factored by 0.4 and applied to the design storms. Losses are generally in the order of 3 to 14 mm for burst initial loss, and 0.6 to 1 mm/hour for continuing loss. Probability neutral burst initial loss values are dependent on the AEP and duration of the design event. An initial loss of 1.5 mm was applied to impervious surfaces. For the PMF event an initial loss of 0 mm and a continuing loss of 0 mm/hr were applied.

8.6. Areal Reduction Factors

Areal Reduction Factors (ARF) were applied in the WBNM model for the design storm events based on ARR2019 (Reference 1). The design rainfall estimates are based on point rainfalls and in reality, the catchment-average rainfall depth will be less. It allows for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms simultaneously over the whole catchment area. The ARF varies with AEP and duration and the resulting matrix of ARFs for the design storms is shown in Table 13. The equations used to derive these reduction factors can be found in Appendix A.

Table 13: Areal Reduction Factors for the Design Storm Events

Duration	AEP							
(min)	50%	20%	10%	5%	2%	1%	0.5%	0.2%
30	0.94	0.94	0.94	0.93	0.93	0.93	0.92	0.92
45	0.95	0.95	0.95	0.94	0.94	0.94	0.93	0.93
60	0.96	0.96	0.95	0.95	0.94	0.94	0.94	0.93
90	0.97	0.96	0.96	0.96	0.95	0.95	0.94	0.94
120	0.97	0.97	0.96	0.96	0.95	0.95	0.94	0.94
180	0.98	0.97	0.97	0.96	0.96	0.95	0.94	0.94
270	0.98	0.98	0.97	0.97	0.96	0.96	0.96	0.95
360	0.99	0.98	0.98	0.98	0.97	0.97	0.97	0.97
540	0.99	0.99	0.99	0.98	0.98	0.98	0.98	0.98
720	0.99	0.99	0.99	0.99	0.98	0.98	0.98	0.98
1080	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
1440	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
1800	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99

8.7. Blockage

There are multiple factors to be considered in assessing the potential for blockage of culverts and bridges. These considerations include:

- the type and mobility of debris that can be washed into the waterway to block the structure or inlet;
- the dimensions of the debris in comparison to the structure;
- dimensions of the structure in relation to the upstream and downstream channels;
- the presence of piers, service crossings, or other obstructions to flow on which debris can accumulate; and
- catchment land-use.

Design blockage factors were adopted based on the ARR2019 guidance for blockage (Reference 15) with consideration of the control inlet dimensions and debris potential. The catchment upstream of The Kilns consists primarily of steep, forested areas. Landslides may occur in this area resulting in the transportation of debris downstream. Between Pittwater Road and Harbord Road the catchment consists of industrial development. The remainder of the catchment consists primarily of medium density urban residential development, with several parks and sporting fields located adjacent to Curl Curl Lagoon. Based on the catchment land-uses an assessment of debris availability, debris mobility and debris transportability was undertaken. The likelihood of debris blockages was deemed to be in the medium category for the Greendale Creek catchment.

The design blockage factors applied to hydraulic structures are detailed in Table 14.

Table 14: Design blockage factors applied at mainstream hydraulic structures

Structure	Type	Design Blockage (%)
Harbord Road GPT (trash rack)	GPT	100
Western Footbridge	Bridge	0
Eastern Footbridge	Bridge	0
Griffin Road Bridge	Bridge	5
Culverts/ Pipes (headwall entrances)	Culvert	25
Inlet Pits	Stormwater Pit	25

Culverts and pipes with headwall entrances were modelled as 25% blocked due to the potential for debris to bridge the structure, blocking the entrance, or become lodged in the barrel. The trash rack on the Harbord Road GPT was modelled as 100% blocked due to the high likelihood for debris to block the structure. Low blockage factors were applied for clear spanning footbridges and bridges in the lower catchment. Sensitivity analysis was undertaken for these blockage assumptions (see Section 10.5).

8.8. Ocean Level Boundary Conditions

Tailwater level assumptions at the downstream ocean boundary do not have a significant influence on peak flood levels in the area of interest due to the perched nature of Curl Curl Lagoon. The major conditions factors controlling flooding in the lower catchment are the lagoon water levels and the level of the sand berm, which is common for a “Group 4” ICOLL. The primary driving factor for flood levels in the lagoon is the berm height rather than the ocean water level or wave conditions, and the design flood modelling approach reflects this, in accordance with the relevant guideline (Reference 16).

When water overtops the berm it erodes, opening the lagoon to the ocean. Once the water has flowed out of the lagoon, tidal forces and wave action begin to push sand back up into the lagoon entrance. This process occurs rapidly at Curl Curl lagoon, and accounts from Council staff indicate that the entrance is often closed again within 24-48 hours. Wave and wind action increases the height of the berm to well above ocean high tide levels. Therefore, the lagoon is closed for a significant majority of the time, and it is the height of the berm that provides the controlling influence over water levels within the lagoon, rather than ocean levels.

The downstream ocean boundary was set to mean sea level (0 mAHD) for design flood analysis.

A Flood Frequency Analysis (FFA) was undertaken for recorded water levels in Curl Curl

Lagoon at Griffin Road Bridge (213426). This analysis provides a reasonable indicator for the variability of the berm height, and the likelihood of a given height being exceeded in a year. The maximum lagoon level in any given year is very close to the maximum berm height, because typically openings occur with only a shallow overtopping depth across the berm.

Design berm heights were set by adopting the corresponding levels determined from the water level FFA. For the PMF event a berm height based on extrapolation past the 1 in 100,000 AEP berm height was adopted. The berm heights adopted for each AEP are shown in Table 15.

Table 15: Catchment Rainfall Event and Corresponding Design Berm Height

Design Event	Berm Height (mAHD)
50% AEP	2.45
20% AEP	2.55
10% AEP	2.61
5% AEP	2.66
2% AEP	2.71
1% AEP	2.75
0.5% AEP	2.78
0.2% AEP	2.83
PMF	3.10

The adopted berm heights typically resulted in design flood levels approximately equal to, or slightly greater than, those determined from the FFA on the downstream side of Griffin Road Bridge. For rare events flood levels were typically higher since these events have faster rates of rise in the lagoon, and produce a greater overtopping depth across the berm. Once the berm is overtopped, the berm is assumed to erode, leaving a 70 m wide channel to the ocean. This erosion was assumed to occur over a period of 6 minutes, based on iterative modelling of the November 2018 historical event.

8.9. Initial Water Level Assumptions

The initial water level in Curl Curl Lagoon was set to 2 mAHD. This initial water level is equal to the level adopted for the 2004 Flood Study (Reference 2) and the initial water level recorded for the November 2018 calibration event. This initial water level is considered to be reasonably typical of lagoon water levels prior to a flood event based on recorded water levels at the gauge. The sensitivity of design flood levels to this assumption was assessed in Section 10.7.

9. DESIGN FLOOD MODELLING RESULTS

Design flood behaviour simulated by the TUFLOW model is presented in the following maps:

- Peak flood depths and levels in Figure D1 to Figure D9;
- Peak flood velocities in Figure D10 to Figure D18;
- Hydraulic hazard based on the FDM (Reference 5) in Appendix E
- Figure E1 to Figure E3;
- Hydraulic hazard based on the Australian Disaster Resilience (ADR) Handbook (Reference 17) in Figure E4 to Figure E6;
- Hydraulic categories (flood function) in Figure E7 to Figure E9.

These results are available in electronic GIS and tabular format. The digital data should be used in preference to the figures in this report as they provide more detail. The maps are intended to provide an overview of the results and should not be relied upon for detailed information at individual properties.

Additional results are presented in the following charts:

- Water level hydrographs at road crossings in Figure F2 to Figure F10 (see Figure F1 for locations); and
- Tables of peak flood levels and flows at key locations in Table 16 to Table 18 below (see Figure 15 for locations).

Discussion of these results is provided in the following sections.

9.1. Flood Behaviour

Much of the upper Greendale Creek catchment is affected by shallow (<0.15 m) overland flow in extreme storm events. This is common for urbanised areas, although in this catchment there several locations where overland flow occurs through property rather than along the road reserves. This is a result of roads often not being aligned with the natural gullies of the upper catchment. The risk to life from this shallow flow is low, and damage to property can generally be minimised provided floor levels are raised relative to surrounding ground levels, and some provision is made to allow overland flow through the properties, rather than blocking it completely.

In the upper catchment there is a relatively flat plateau to the north of Warringah Road and water ponds at a sag point in McKillop Road. To the south of Warringah Road, the upper reach of Greendale Creek forms through the joining of several small creek lines. This creek discharges into a 2.5 m (W) x 1.2 m (H) trunk drainage line just upstream of Consul Road. The capacity of this trunk line at the upstream end is exceeded in a 10% AEP event, causing overland flow that follows the drainage line to Pittwater Road. Water flows through properties as well as along Gulliver Street, Alfred Road and eventually ponding on Pittwater Road. Pittwater Road also collects shallow overland flows from the catchment to the north.

Downstream of Pittwater Road, there is more significant flooding through the Brookvale industrial area. The trunk drainage line (now a 1.5 m diameter pipe) discharges into a small open channel downstream of Winbourne Road, before being carried by pipes (2 x 1.8 m diameter) to just downstream of Ethel Avenue. From Ethel Avenue, water is discharged through a series of pipes and small open channels into the Greendale Creek channel immediately downstream of Harbord Road. Through the industrial area, significant flood depths can occur at Mitchell Road (and at properties just downstream), at the Winbourne Estate (at the end of Chard Road) and through properties and roads just upstream of the Greendale Creek channel (along Ada Avenue, Ethel Avenue and Harbord Road). This ponding occurs in events as small as the 50% AEP event.

The open channel portion of Greendale Creek downstream of Harbord Road is approximately 30 m wide and conveys flows towards Curl Curl Lagoon. Flows up to and including the 0.2% AEP are contained within the channel. The main body of the lagoon is between the rock weir and Griffin Road Bridge. Downstream of the bridge the lagoon discharges through North Curl Curl Beach into the ocean. This is dependent on whether the lagoon is open or closed, dictated by a sand bar that forms at the entrance. Within Curl Curl Lagoon, the water level is primarily influenced by this berm height. Different berm heights have been adopted for different design flood events, which dictates the peak flood levels within the lagoon.

Downstream of Harbord Road, there are numerous stormwater lines and overland flow paths that discharge into Greendale Creek and Curl Curl Lagoon on both the northern and southern sides. On the southern side, ponding occurs in the vicinity of the Harbord Bowling Club and around Weldon Oval in events as small as the 50% AEP. Flooding also occurs at the rear of properties along Stirgress Avenue and Stewart Avenue. Flooding from the lagoon also affects the Adam Street Reserve. On the northern side, ponding occurs in three locations along Abbott Road between Harbord Road and Pitt Road. To the east of this, there are four main flow paths that traverse residential properties and discharge into Greendale Creek. These are located just to the west of Playfair Road, between Ross Street and Grainger Avenue, between Spring Street and Blackwood Road, and through Surf Reserve. In several cases for these local drainage lines into the lagoon, overland flow is obstructed from reaching the lagoon due to the filled playing fields being higher than the upstream ground levels.

9.2. Tables of Peak Flood Levels, Depths and Flows

A tabulated summary of peak flood levels, depths and flows at selected locations as shown in Figure 15 are detailed in Table 16, Table 17 and Table 18, respectively. These key locations coincide with the key locations used for the sensitivity analysis discussed in Section 10.

Table 16: Peak Flood Levels (mAHD) at Key Locations

ID	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
H01	McKillop Road	128.83	128.98	129.06	129.14	129.21	129.28	129.32	129.38	129.69
H02	Upstream 44 Consul Road	30.27	30.50	30.70	30.87	31.00	31.03	31.04	31.11	31.66
H03	Consul Road	29.95	29.98	30.40	30.59	30.69	30.75	30.78	30.83	31.19
H04	Gulliver Street	26.11	26.28	26.33	26.35	26.41	26.45	26.48	26.53	26.90
H05	West of Brookvale Oval (Pittwater Road)	22.51	22.63	22.68	22.71	22.75	22.78	22.81	22.86	23.30
H06	Pittwater Road	18.98	19.07	19.12	19.16	19.22	19.26	19.28	19.32	19.68
H07	Winbourne Road	15.58	15.66	15.69	15.72	15.73	15.75	15.77	15.80	16.08
H08	Upstream Chard Road	10.53	10.96	11.45	11.91	12.22	12.39	12.48	12.61	13.58
H09	Ethel Avenue	5.93	6.13	6.32	6.46	6.58	6.69	6.77	6.91	8.23
H10	Upstream Harbord Road	4.98	5.35	5.82	6.05	6.20	6.32	6.41	6.54	7.67
H11	Harbord Road	4.97	5.01	5.04	5.06	5.12	5.20	5.26	5.34	6.69
H12	Downstream Harbord Road	3.63	3.87	4.01	4.13	4.24	4.37	4.49	4.67	6.60
H13	Downstream Harbord Road GPT (Gross Pollutant Trap)	3.54	3.78	3.91	4.03	4.13	4.27	4.37	4.55	6.38
H14	Downstream Rock Weir	2.49	2.60	2.68	2.76	2.80	2.86	2.92	2.99	4.66
H15	Upstream Griffin Road	2.49	2.60	2.67	2.71	2.77	2.82	2.85	2.91	4.60
H16	Downstream Griffin Road	2.48	2.60	2.65	2.67	2.77	2.79	2.79	2.86	4.46
H17	Upstream Berm	2.41	2.49	2.56	2.64	2.63	2.71	2.77	2.78	3.04
H18	Bennett Street	9.39	9.41	9.42	9.43	9.46	9.47	9.48	9.49	9.58
H19	Mitchell Road	9.98	10.00	10.13	10.23	10.30	10.37	10.41	10.47	11.05
H20	Pitt Road	13.35	13.36	13.39	13.40	13.45	13.46	13.47	13.50	13.64
H21	Abbott Road	4.01	4.06	4.09	4.11	4.13	4.15	4.16	4.19	4.64
H22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	3.71	3.71	3.72	3.74	3.74	3.75	3.76	3.76	4.66
H23	Upstream Reub Hudson Oval (Abbott Road)	10.06	10.23	10.36	10.45	10.52	10.59	10.63	10.68	10.93
H24	Downstream Northern Beaches Secondary College	9.96	10.21	10.34	10.45	10.54	10.60	10.64	10.69	10.96
H25	Manuela Place	5.08	5.24	5.30	5.33	5.33	5.35	5.37	5.39	5.57
H26	Upstream Western Footbridge	3.16	3.39	3.51	3.61	3.70	3.82	3.93	4.09	5.52
H27	Downstream Western Footbridge	3.16	3.38	3.50	3.60	3.68	3.80	3.89	4.05	5.48
H28	Upstream Eastern Footbridge	2.51	2.65	2.71	2.80	2.88	2.93	2.98	3.09	4.73
H29	Downstream Eastern Footbridge	2.49	2.62	2.68	2.77	2.82	2.86	2.92	3.01	4.68

Table 17: Peak Flood Depths (m) at Key Locations

ID	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
D01	McKillop Road	0.31	0.46	0.55	0.62	0.69	0.76	0.80	0.86	1.21
D02	Upstream 44 Consul Road	1.47	1.70	1.90	2.07	2.20	2.23	2.24	2.31	3.06
D03	Consul Road	0.00	0.03	0.45	0.64	0.74	0.81	0.84	0.88	1.40
D04	Gulliver Street	0.00	0.17	0.22	0.24	0.30	0.34	0.37	0.42	0.93
D05	West of Brookvale Oval (Pittwater Road)	0.26	0.38	0.43	0.46	0.50	0.53	0.56	0.61	1.23
D06	Pittwater Road	0.51	0.59	0.64	0.69	0.75	0.78	0.81	0.85	1.35
D07	Winbourne Road	0.12	0.20	0.23	0.25	0.27	0.29	0.30	0.33	0.71
D08	Upstream Chard Road	1.59	2.02	2.51	2.97	3.28	3.45	3.54	3.67	4.87
D09	Ethel Avenue	0.40	0.61	0.80	0.93	1.05	1.16	1.25	1.38	2.85
D10	Upstream Harbord Road	1.90	2.28	2.74	2.98	3.13	3.25	3.34	3.46	4.72
D11	Harbord Road	0.07	0.12	0.14	0.16	0.23	0.31	0.36	0.45	1.91
D12	Downstream Harbord Road	2.80	3.05	3.18	3.30	3.41	3.55	3.66	3.84	5.88
D13	Downstream Harbord Road GPT (Gross Pollutant Trap)	2.43	2.67	2.81	2.93	3.03	3.16	3.27	3.45	5.37
D14	Downstream Rock Weir	1.82	1.93	2.01	2.09	2.13	2.19	2.25	2.32	3.99
D15	Upstream Griffin Road	1.92	2.03	2.10	2.15	2.21	2.25	2.28	2.35	4.03
D16	Downstream Griffin Road	2.38	2.49	2.55	2.57	2.67	2.68	2.69	2.76	4.36
D17	Upstream Berm	1.69	1.76	1.84	1.92	1.91	1.99	2.05	2.06	2.32
D18	Bennett Street	0.33	0.34	0.36	0.37	0.39	0.40	0.41	0.42	0.57
D19	Mitchell Road	0.02	0.05	0.18	0.28	0.35	0.41	0.45	0.51	1.14
D20	Pit Road	0.16	0.17	0.20	0.22	0.26	0.27	0.28	0.31	0.58
D21	Abbott Road	0.06	0.11	0.14	0.16	0.18	0.20	0.21	0.23	0.69
D22	Upstream Curl Curl Youth and Community Centre (Abbott Road)	0.24	0.25	0.26	0.27	0.28	0.29	0.29	0.30	1.19
D23	Upstream Reub Hudson Oval (Abbott Road)	0.13	0.29	0.43	0.52	0.59	0.65	0.69	0.74	1.04
D24	Downstream Northern Beaches Secondary College	0.17	0.42	0.55	0.67	0.76	0.82	0.86	0.91	1.23
D25	Manuela Place	0.26	0.42	0.48	0.51	0.51	0.53	0.55	0.57	0.85
D26	Upstream Western Footbridge	1.99	2.22	2.34	2.44	2.53	2.65	2.76	2.92	4.39
D27	Downstream Western Footbridge	2.32	2.54	2.66	2.76	2.85	2.96	3.05	3.21	4.68
D28	Upstream Eastern Footbridge	2.00	2.14	2.20	2.29	2.37	2.42	2.48	2.59	4.22
D29	Downstream Eastern Footbridge	2.02	2.15	2.21	2.30	2.35	2.39	2.45	2.54	4.21

Table 18: Peak Flood Flows (m^3/s) at Key Locations

ID	Location	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Q01	Upstream of The Kilns (West)	2.6	3.2	3.6	4.0	5.1	5.7	6.2	6.8	12.8
Q02	Upstream of The Kilns (East)	0.8	1.0	1.2	1.4	1.7	2.0	2.5	3.1	11.9
Q03	Upstream Consul Road	4.0	5.2	6.1	7.0	8.5	10.0	10.9	12.3	29.8
Q04	Consul Road	0.0	0.1	0.6	1.7	3.2	4.7	5.6	7.1	25.9
Q05	Gulliver Street	0.9	3.8	5.1	5.9	7.3	9.4	10.7	13.7	45.9
Q06	Downstream Winbourne Road	6.4	7.5	8.2	8.9	9.4	10.1	10.6	11.3	23.6
Q07	Upstream Harbord Road	13.6	15.8	17.8	19.8	21.9	24.1	26.1	29.5	87.9
Q08	Downstream Harbord Road	15.8	18.9	21.6	24.3	26.9	30.2	33.1	38.2	149.9
Q09	Upstream Western Footbridge	17.4	21.9	25.1	28.0	30.2	33.7	36.6	41.5	134.9
Q10	Upstream Eastern Footbridge	21.6	28.6	31.9	35.0	37.7	41.9	45.4	51.8	198.7
Q11	Downstream Rock Weir	27.8	35.5	38.3	41.1	46.7	50.7	54.7	62.3	238.7
Q12	Downstream Griffin Road	49.0	55.9	61.3	65.6	67.5	73.4	76.6	81.7	295.6
Q13	Curl Curl Lagoon Berm	55.3	64.2	67.8	87.9	78.1	90.0	104.1	98.1	316.9
Q14	Adams Street	0.5	0.6	0.8	0.9	1.4	1.6	1.7	2.0	7.3
Q15	Bennett Street	0.6	0.9	1.1	1.4	2.1	2.4	2.7	3.1	7.6
Q16	Manuela Place	0.0	0.0	0.1	0.2	0.2	0.3	0.4	0.6	2.8
Q17	Pit Road	0.4	0.5	0.6	0.7	1.2	1.4	1.6	1.9	5.1
Q18	Abbott Road	1.0	2.0	2.8	3.7	4.7	5.9	6.9	8.5	27.2
Q19	Upstream Community Centre (Abbott Rd)	0.6	0.7	1.0	1.4	1.5	1.7	1.9	2.3	8.9
Q20	Upstream Reub Hudson Oval (Abbott Rd)	0.2	0.4	0.6	0.8	1.1	1.2	1.4	1.7	6.5
Q21	Downstream Northern Beaches Secondary College	0.0	0.0	0.1	0.1	0.2	0.3	0.4	0.5	5.1

9.3. Hydraulic Hazard Categorisation

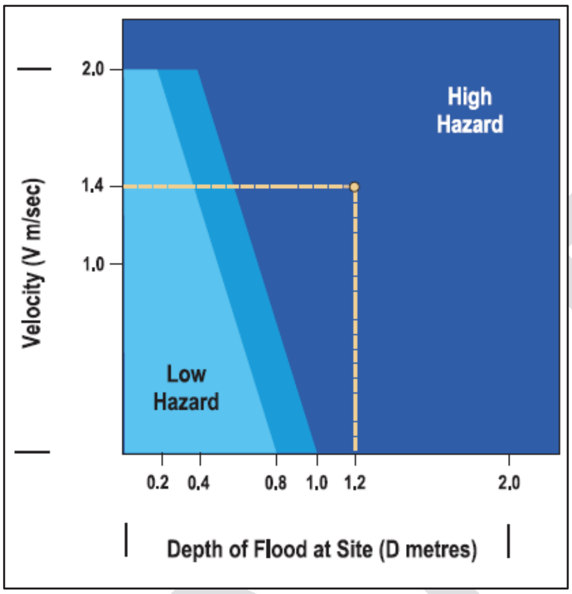
Hydraulic hazard is a measure of potential risk to life and property damage from flooding and is typically determined by considering the depth and velocity of floodwaters. In recent years, there have been a number of developments in the classification of hazards. Research has been undertaken to assess the hazard to people, vehicles and buildings based on flood depth, velocity and velocity depth product.

Provisional hazard categories have been determined for the Greendale Creek catchment by two methods - one in accordance with the NSW FDM (Reference 5), and the other in accordance with the Australian Disaster Resilience Handbook Collection (Reference 17). Each method of provisional flood hazard categorisation is discussed below.

9.3.1. Floodplain Development Manual

Appendix L of the NSW FDM (Reference 5) provides one method for hydraulic hazard, which is shown in Diagram 3. In this study, the transition zone was considered to be high hazard.

Diagram 3: Provisional “L2” Hydraulic Hazard Categories (Reference 5)



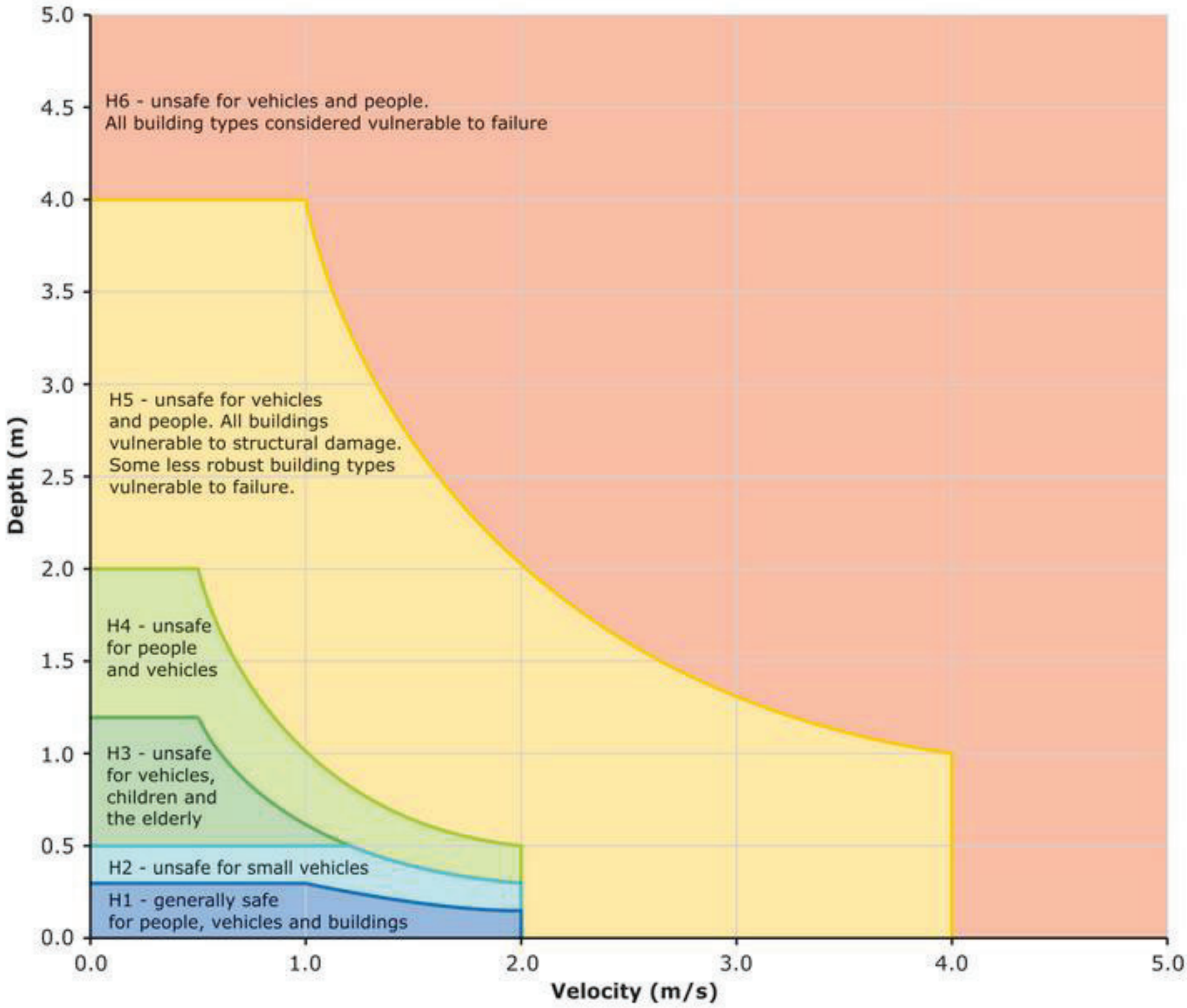
The hydraulic hazard utilising the FDM categorisation is mapped on Appendix E Figure E1 to Figure E3 for the 5% AEP, 1% AEP and PMF events respectively. The FDM hazard categorisation has been included for applicability to existing council policy documents that may refer to this hazard classification.

In the 5% and 1% AEP events, high hazard areas are generally restricted to the creek channels and Curl Curl Lagoon, with some areas of deeper ponding on roads also classified as high hazard. In the PMF event, high hazard covers a much larger area, including areas adjacent to Curl Curl Lagoon, most of the roads within the Brookvale industrial area and other roads that have a high conveyance of flows throughout the catchment.

9.3.2. Australian Disaster Resilience Handbook Collection

The Australian Disaster Resilience (ADR) Handbook Collection deals with floods in Handbook 7 (Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia). The supporting guideline 7-3 (Reference 17) contains information relating to the categorisation of flood hazard. A summary of this categorisation is provided in Diagram 4.

Diagram 4: General Flood Hazard Vulnerability Curves (ADR)



This classification provides a more detailed distinction and practical application of hazard categories, identifying the following 6 classes of hazard:

- H1 – Generally safe for vehicles, people and buildings;
- H2 – Unsafe for small vehicles;
- H3 – Unsafe for all vehicles, children and the elderly;
- H4 – Unsafe for all people and all vehicles;
- H5 – Unsafe for all people and all vehicles. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure; and
- H6 – Unsafe for all people and all vehicles. All building types considered vulnerable to failure.

The hazard maps using the ADR classification are shown in Figure E4 to Figure E6 for the 5% AEP, 1% AEP and PMF events respectively. In the 5% AEP and 1% AEP events, H4 and H5 hazard are typically contained to the creek channel and Curl Curl Lagoon. In areas of deeper ponding, the hazard is H2 and H3, with the remaining areas affected by overland flooding being H1. In the PMF event the hazard reaches H6 in the Greendale Creek channel and H5 in the Brookvale industrial area.

9.4. Flood Function

Identification of flood function involves mapping the floodplain to indicate which areas are most important for the conveyance of floodwaters, and the temporary storage of floodwaters. This can help in planning decisions about which parts of the floodplain are suitable for development, and which areas need to be left as-is to ensure that flooding impacts are not worsened compared to existing conditions. Typically, development within floodway or flood storage areas would be likely to push water into other areas and redistribute the flood risk, unless the development is carefully designed to avoid these impacts.

The 2005 NSW Government's FDM (Reference 5) defines three hydraulic categories which can be applied to different areas of the floodplain depending on the flood function:

- Floodways;
- Flood Storage; and
- Flood Fringe.

Floodways are areas of the floodplain where a significant discharge of water occurs during flood events and by definition, if blocked would have a significant effect on flood levels and/or distribution of flood flow. Flood storages are important areas for the temporary storage of floodwaters and if filled would result in an increase in nearby flood levels and the peak discharge downstream may increase due to the loss of flood attenuation. The remainder of the floodplain is defined as flood fringe.

There is no quantitative definition of these three categories or accepted approach to differentiate between the various classifications. The delineation of these areas is somewhat subjective depending on knowledge of an area and flood behaviour, hydraulic modelling and previous experience in categorising flood function. The method defined by Howells *et al* (Reference 18), relies on combinations of velocity and depth criteria to define the floodway.

For this study, hydraulic categories were defined by the following criteria, which correspond in part with the criteria proposed by Howells *et al*, 2003 (Reference 18):

- Floodway is defined as areas where:
  - the peak value of velocity multiplied by depth (V x D) > 0.25 m^2/s **AND** peak velocity > 0.25 m/s, OR
  - peak velocity > 1.0 m/s

The remainder of the floodplain is either Flood Storage or Flood Fringe,

- Flood Storage comprises areas outside the floodway where peak depth > 0.2 m; and
- Flood Fringe comprises areas outside the Floodway where peak depth < 0.2 m.

Figure E7 to Figure E9 show the provisional hydraulic categorisations for the Greendale Creek catchment for the 5% AEP, 1% AEP and PMF events, respectively. In the 5% AEP and 1% AEP events the floodway is generally restricted to the Greendale Creek channel and some of the roads that have a high conveyance of flows. Flood storage areas include the lagoon and areas of deep ponding adjacent to the creek and through the Brookvale industrial area. In the PMF event, the floodway and flood storage areas are extensive, covering much of the Brookvale industrial area and areas surrounding Greendale Creek and Curl Curl Lagoon.

9.5. Information to Support Decisions on Activities in the Floodplain and Managing Flood Risk

It is considered good practice to permit land use and development that is compatible with the nature of flooding in a particular area. The following sections provide information that is relevant to support decisions on activities in the floodplain and managing flood risk.

9.5.1. Road Inundation

An analysis of road inundation was undertaken at key locations (Figure F1) in the study area. Stage hydrographs showing the depths at selected roads and crossings of Greendale Creek and Curl Curl Lagoon are shown in Figure F2 to Figure F10.

9.5.2. Pipe Capacity Assessment

The design flood results were used to determine how frequently the stormwater pipe system capacity is likely to be exceeded throughout the catchment. Defining the capacity of a pipe is not straightforward, as it depends on multiple factors including shape, the flow regime (e.g. upstream or downstream controlled), inlet and outlet connection, pipe grade, and other factors.

TUFLOW provides output indicating the proportion of the cross-section area of a pipe that has flow in it. For this assessment, pipes were assumed to be “full” when the flow area was equal to or in excess of 85% of the pipe’s cross-sectional area. This is the point at which circular pipes tend to be close to their most efficient, since at 100% of cross-sectional area the additional friction from the top of the pipe reduces pipe conveyance. Similarly, box culverts designed for a supercritical flow regime will typically be designed for free surface flow approximately 80% of the depth of the culvert, as when flow touches the pipe soffit it will typically “trip” the flow regime to become pressurised, resulting in lower capacity, depending on the pipe grade. Additionally, due to energy losses associated with adjoining pits, inlets, bends etc., some culverts may never reach “100% full” capacity by waterway area, although they may be 90% full for a range of design events (e.g. from the 5% AEP through to the PMF). In such circumstances, it is informative to know the design storm for which the pipe is almost at its maximum capacity.

Figure 16 shows the results of the pipe capacity assessment for the modelled range of design events. A large proportion (approximately 70%) of the pipes are full in the 50% AEP event.

9.5.3. Flood Planning Constraint Categories

Guideline 7-5 of the Australian Disaster Resilience Handbook Collection (Reference 19) recommends using Flood Planning Constraint Categories (FPCCs) to better inform land use planning activities. These categories condense the wealth of flood information produced in a flood study and classify the floodplain into areas with similar degrees of constraint. These FPCCs can be used in high level assessments of land use planning to inform and support decisions. For detailed land use planning activities, it is recommended that the flood behaviour across the range of flood events be considered, depending on the level of constraint.

Council’s existing planning policies and framework do not reference FPCCs, but they can still provide value for Council’s internal strategic planning activities to understand flood constraints. Specific developments should be assessed on a merits basis taking into consideration the full range of flood behaviour possible at that location and the type of development proposed.

The following range of flood map outputs were considered and combined to develop a Flood Planning Constraint Category Map for the study area:

- Flood Extents,
- Hydraulic Hazard,
- Flood Function,
- Flood Emergency Response Classifications for Communities, and
- Flood Planning Area.

The methodology adopted was to delineate the floodplain into four planning categories, consistent with the approach from (Reference 19), adopting the 1% AEP as the defined flood event, and the 0.2% (1 in 500) AEP as the larger event. The definition for each FPCC category is provided below:

- FPCC1: Flow conveyance (floodway) and storage areas in the 1% AEP and H6 hazard areas in the 1% AEP. The majority of developments and uses have adverse impacts on flood behaviour. Consider limiting uses and development to those compatible with the flood behaviour. Development involving structures or fill in these areas is likely to produce adverse flood impacts in other areas.
- FPCC2: Flow conveyance (floodway) areas in the 0.2% AEP, H5 hazard category in the 1% AEP, H6 in the 0.2% AEP. Consider compatibility of developments and users with rare flood flows in the area. Many uses and developments will be vulnerable to flood hazard. Consider limiting new uses to those compatible with the flood hazard. Consider treatments to reduce the flood hazard which will not adversely affect flood behaviour. Consider evacuation difficulties.
- FPCC3: Outside FPCC2, but within the Flood Planning Area. Hazardous conditions may exist creating issues for vehicles, people and buildings. Standard land-use and development controls aimed at reducing damage and exposure of the development to flooding in the 1% AEP are likely to be suitable. Consider the need for additional conditions for emergency response facilities, key community infrastructure and vulnerable users within these areas due to potential access difficulties.
- FPCC4: Outside FPCC3, but within the PMF extent. Consider the need for special development conditions for emergency response facilities, key community infrastructure and land uses with vulnerable users.

Any changes in land use or new developments should be compatible with the nature of flooding in the area. The information contained in the flood study regarding the flood hazard, flood function and evacuation potential should be used in land use planning activities to ensure that proposed land uses do not increase the flood risk to people or property. The results obtained using the above methodology are mapped on Figure E10

9.5.4. Flood Planning Area

9.5.4.1. Background

Land use planning is an effective means of minimising flood risk and damages from flooding. Land use planning for flooding can be achieved through the use of:

- A Flood Planning Area (FPA), which identifies land that is subject to flood related development controls; and
- A Flood Planning Level (FPL), which identifies the minimum floor level applied to development proposals within the FPA.

Defining FPAs and FPLs in urban areas can be complicated by the variability of flow conditions between mainstream and local overland flow. Traditional approaches developed for riverine or "mainstream" flow areas often cannot be applied in steeper urban overland flow areas. Additionally, defining the area of flood affectation due to overland flow (which by its nature includes shallow flow) involves determining at which point flow is significant enough to be classified as "flooding" rather than just a drainage or local runoff issue. In some areas of overland flow, the difference in peak flood level between events of varying magnitude can be so minor that applying the typical freeboard can result in an FPL greater than the PMF level.

The FPA should include properties where development would result in impacts on flood behaviour in the surrounding area and in areas of high hazard where there is a risk to safety or life. The FPL is determined in addition to this with the purpose of decreasing the likelihood of over-floor flooding of buildings.

The Floodplain Development Manual (Reference 5) suggests that the FPL generally be based on the 1% AEP event plus an appropriate freeboard (typically 0.5 m). However, it also recognises that different freeboards may be deemed appropriate due to local conditions provided adequate justification is provided.

Further consideration of flood planning areas and levels is typically undertaken as part of the Floodplain Risk Management Study to determine what should be included in the Floodplain Risk Management Plan.

9.5.4.2. Methodology

The methodology used for defining the flood planning area is consistent with that adopted in a number of similar studies throughout the Sydney metropolitan area. It divides the flood area between "mainstream" and "overland" flooding areas using the following criteria:

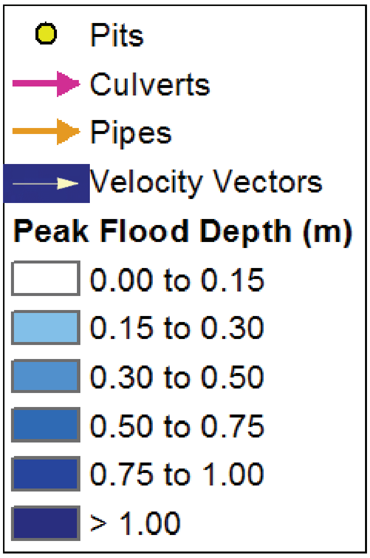
- Mainstream flooding: Areas along the main creeks or trunk drainage alignment, where flow is sufficiently deep and there is sufficient relief that freeboard can be added to the flood surface and the extent then "stretched" to include adjacent land. The mainstream part of the study was defined as Greendale Creek downstream of Harbord Road, including Curl Curl Lagoon. The FPA along this reach was defined as the peak flood level plus 0.5 m freeboard, with the level extended perpendicular to the flow direction either side of the flow path.

- Overland flooding: For overland flow areas, addition of freeboard and stretching generally produces an over-estimate of the land subject to flood risk, because the stretching extends across land in a way that would not actually occur even with significant additional flow from a much larger storm, and may even extend beyond the modelled PMF extent. It is therefore appropriate to use a modelled design flood event larger than the 1% AEP event to account for the uncertainty in the results, instead of adding freeboard and stretching. The advantage of this approach is that it includes consideration of flow momentum from actual model results. In overland flow areas, it was considered appropriate to use the filtered 0.2% AEP extent as the preliminary definition of the Flood Planning Area (FPA). The following filters have been applied to the 0.2% AEP event:
    - **Depth Filter** – Exclude results below 150 mm depth;
    - **Velocity-Depth Filter** – Include results if the Velocity x Depth product > 0.3 m^2/s (even if previously excluded by the Depth Filter); and
    - **Small Pond Filter** – Remove isolated ‘puddles’ or ‘orphans’ smaller than 100 m^2.

Figure 17 identifies the extent of the preliminary FPA (combined mainstream and overland) developed using the methodology above.

9.6. Descriptions of Hot Spots

Diagram 5: Legend for Hot Spot Diagrams

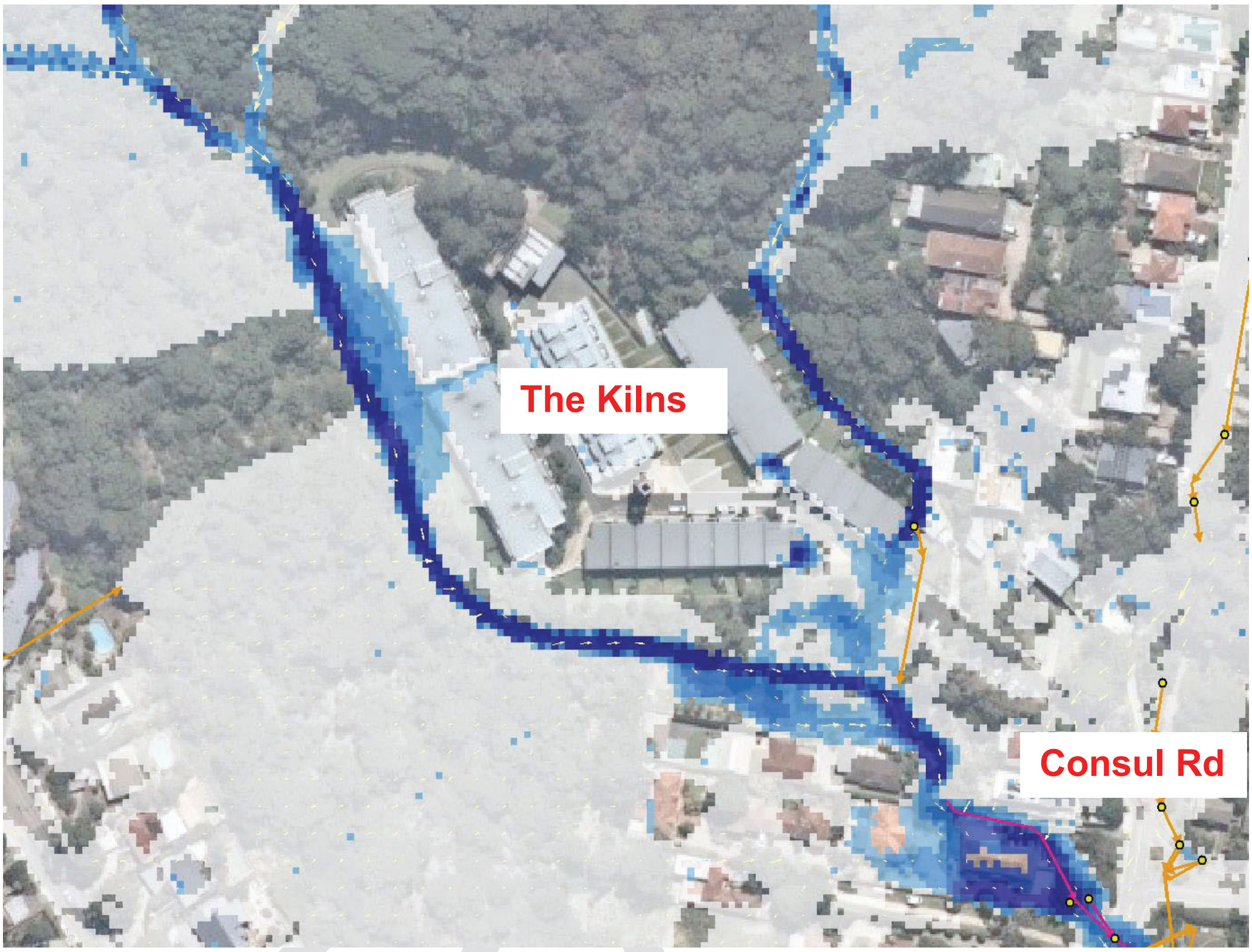


A description of the flow behaviour at locations with the most significant flood risk, or flooding “hot spots” is provided below.

The information shown on the diagram for each hot-spot is as per the legend shown in Diagram 5 to the left. The flood depths indicated on the diagrams in this section are for the 1% AEP event.

9.6.1. The Kilns to St Augustine's College

Diagram 6: "The Kilns" at 48A Consul Road



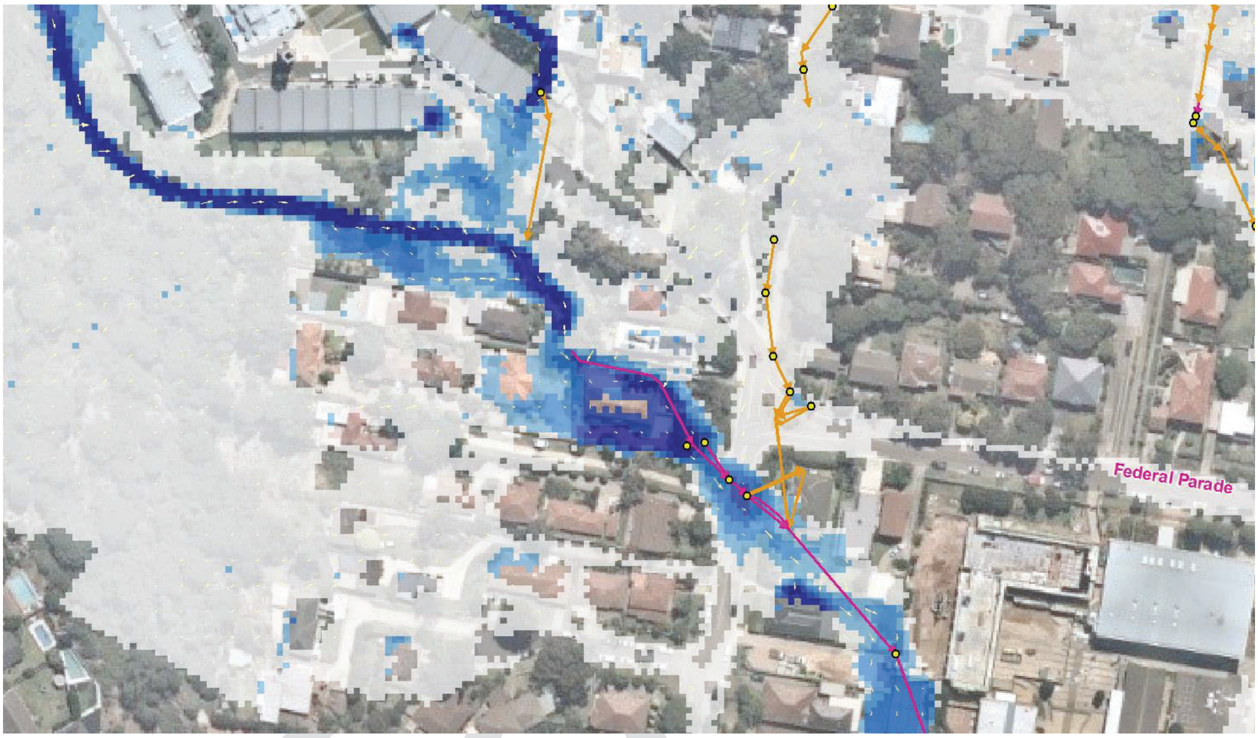
"The Kilns" development lies between two streams that originate at the top of the escarpment at Warringah Road. Each of the streams are intercepted by man-made channels and diverted around the development, before joining on the downstream side and flowing through other properties towards Consul Road (Diagram 6). It is likely that prior to the construction of the brick-making facilities, the creeks went through the site, and the area was filled for the industrial activity.

Photographs of the western flow path are shown in Appendix B, Photo B1 to Photo B5. Photographs of the eastern flow path and cutoff wall are shown in Photo B6 to Photo B10. Photo B11 to Photo B16 show the Greendale Creek channel at the downstream outlet from the Kilns as it flows through private property to Consul Road.

This flow path has the potential to inundate properties on the western side of The Kilns that front onto the channel. Flooding reportedly occurred in the past when there was a landslide of material from the escarpment into the channel. Residents report that there have been fewer issues since the landslide material was removed, but modelling indicates the channel does not have 1% AEP capacity, and in this event some overland flow into the yards and possibly above floor will occur at this location.

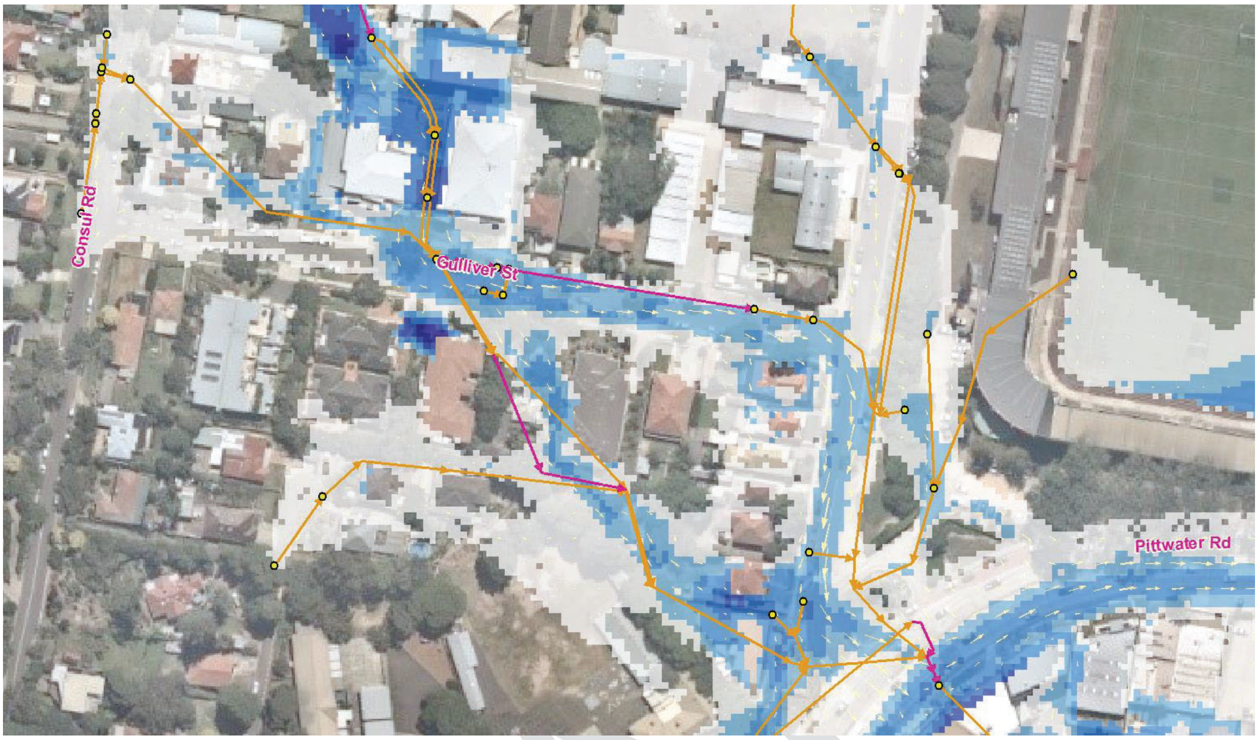
Downsteam of the Kilns the channel flows through low lying properties, particularly 44, 46 and 48 Consul Road. There is a local sag point west of Consul Road, and floodwaters will pond to significant depths in 44 Consul Road when the pipe capacity is exceeded (between 20% AEP and 10% AEP capacity). When flows exceed the Consul Road pipe capacity, they overtop Consul Road and flow south-easterly through 35-47 Consul Road, and then through St Augustine's College (see Diagram 7).

Diagram 7: Flow path from the Kilns to St Augustine's College



9.6.2. St Augustine's College to Pittwater Road

Diagram 8: Flow path from St Augustine's College to Pittwater Road



The overland flow path through St Augustine's college follows the same path as the below-ground trunk drainage network (which has 50% AEP to 20% AEP capacity). The flow path exits St Augustine's College at Gulliver Street, where there is a split in both the sub-surface drainage network and the overland flow paths (Diagram 8). The original flow path continues wouth-eastwards through private property, with some flow being diverted along Gulliver Street and then Alfred Street. These flow paths recombine at the sag point on the bend in Pittwater Road south of Brookvale Oval.

Pictures of the properties and flow paths along Gulliver Street are shown in Appendix B (Photo B17 to Photo B20

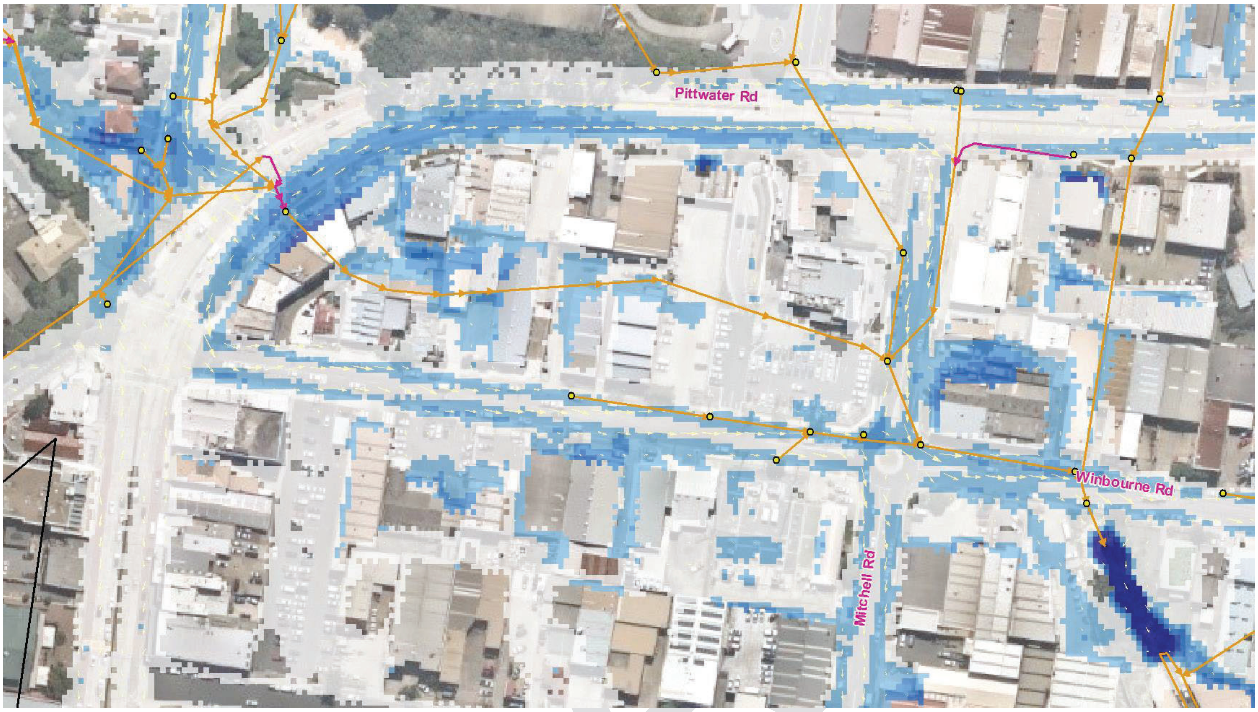
9.6.3. Pittwater Road to Winbourne Road

In the vicinity of Brookvale Oval, there is a bend in Pittwater Road. The road levels are lower on the inside of the bend, towards the south-east, and at this location there is an obstruction to flow from a continuous row of commercial properties (712 to 718 Pittware Road). The sub-surface trunk drain passes under 712 Pittwater Road, and then through the rear of properties on the northern side of Winbourne Road (Diagram 9). Modelling indicates this trunk drain reaches capacity in a 50% AEP event or smaller. Events exeeding this capacity will cause ponding of water on Pittwater Road, until it reaches a depth sufficient to flow around the commercial properties, with a three-way split:

- down Winbourne Road;
- through the gap between 718 and 724 Pittwater Road; and
- easterly along Pittwater Road to Mitchell Road.

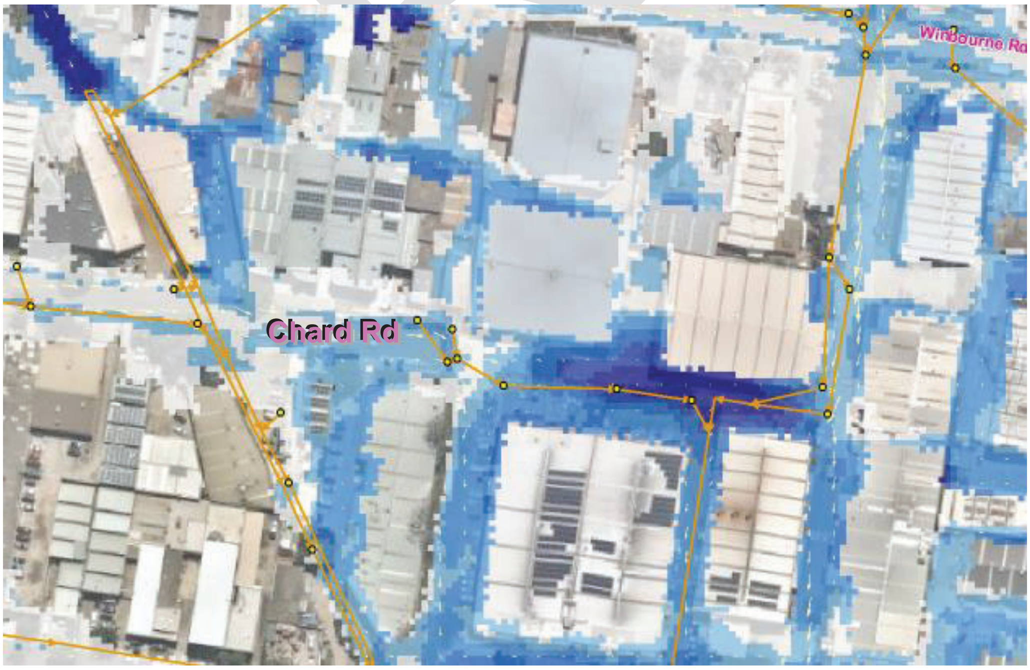
These flow paths recombine near the intersection of Mitchell Road and Winbourne Road, where there is a short section of open channel running south-east from Winbourne Road (see Photo B25 to Photo B27).

Diagram 9: Flow path from Pittwater Road to Winbourne Road



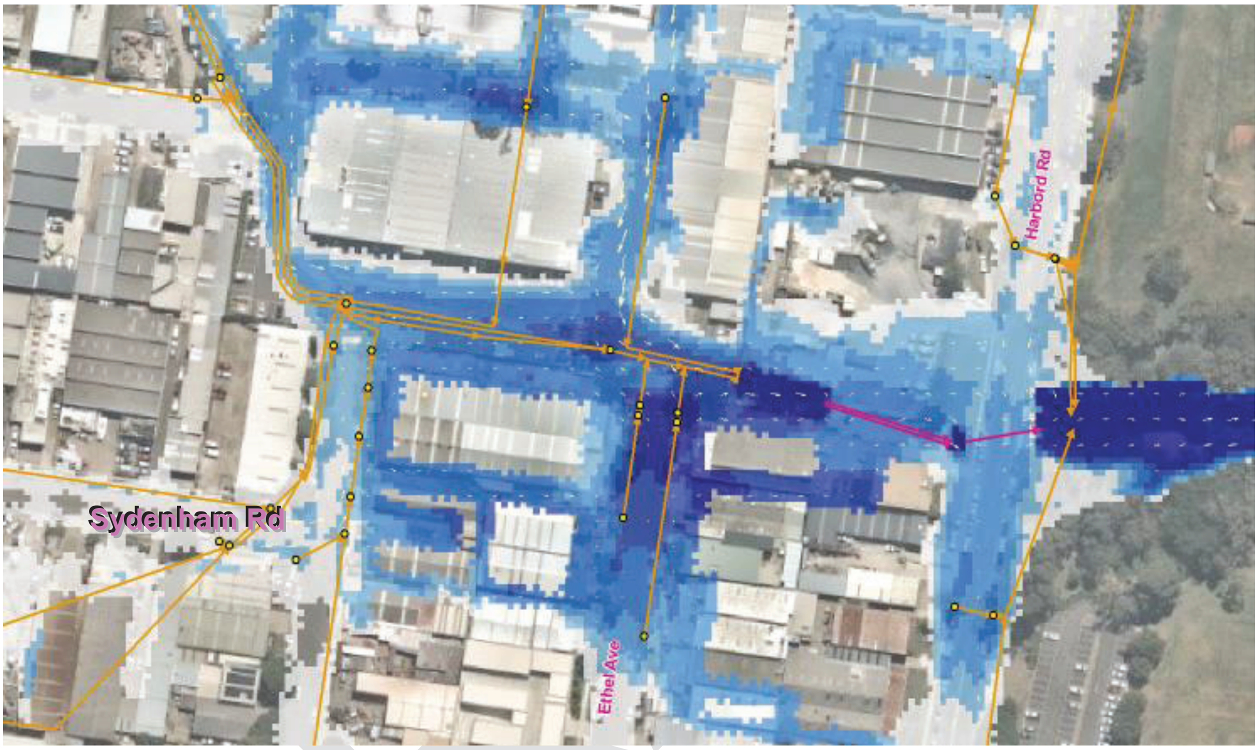
9.6.4. Winbourne Road to Harbord Road

Diagram 10: Sag Point at eastern end of Chard Road



In a 1% AEP event, modelling indicates there will be widespread overland flooding throughout the industrial poperties from downstream of the open channel near Winbourne Road to the open channel at Harbord Road. The overland flow would occur through or around most buildings in this area, not just those in the vicinity of the stormwater network. The most notable areas of significant flooding depth are the low lying area at the eastern end of Chard Road (Diagram 10), and the area between the eastern end of Sydenham Road and the Harbord Road culverts (Diagram 11, also Photo B28 to Photo B30). The pipe drainage capacity throughout this area is generally exceeded in a 50% AEP or 20% AEP event.

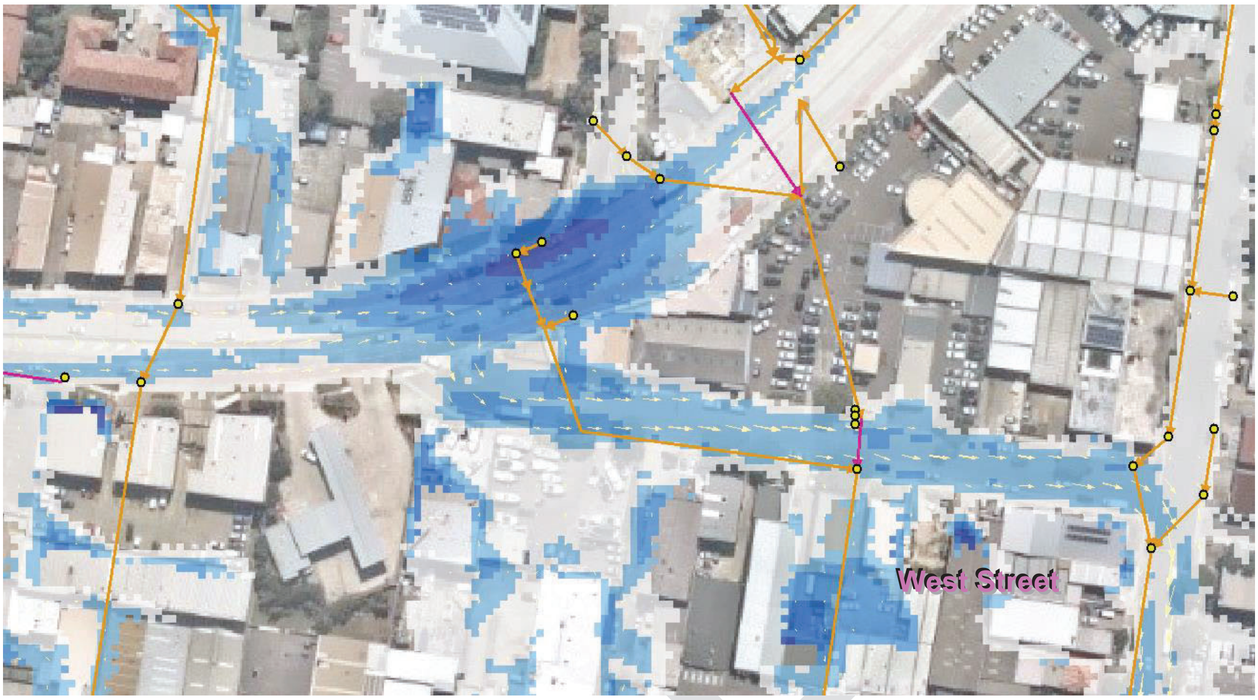
Diagram 11: Overland flow paths in Industrial Area near Harbord Road



9.6.5. Pittwater Road at West Street

There is a local sag point in Pittwater Road at West Street. When the stormwater pipe capacity is exceeded (less than 50% AEP capacity), floodwaters will pond in Pittwater Road to depths of up to 0.8 m in the 1% AEP event, with overland flow exiting the sag point via West Street and continuing across the main trunk drainage line towards Carter Road (Diagram 12). Shallow overland flow is likely occur through properties on West Street, Carter Road and Winbourne Road along this flow path.

Diagram 12: Overland flow paths at Pittwater Road near West Street



9.6.6. Pittwater Road at West Street

Diagram 13: Overland flow paths from Pitt Road to Abbott Road

There are several stormwater drainage lines running downhill from Pitt Road to Abbott Road, which flow through private property rather than along the road network. Three of these drainage lines are depicted in Diagram 13. The capacity of these lines varies, with some less than 50% AEP capacity and some large enough to convey the 1% AEP flow. When flow exceeds the pipe capacity along these lines, overland flow is expected to occur through the properties.

9.6.7. Playing Fields South of Greendale Creek

Diagram 14: Low-lying terrain between Stirgess Avenue and Weldon Oval / Stirgess Reserve

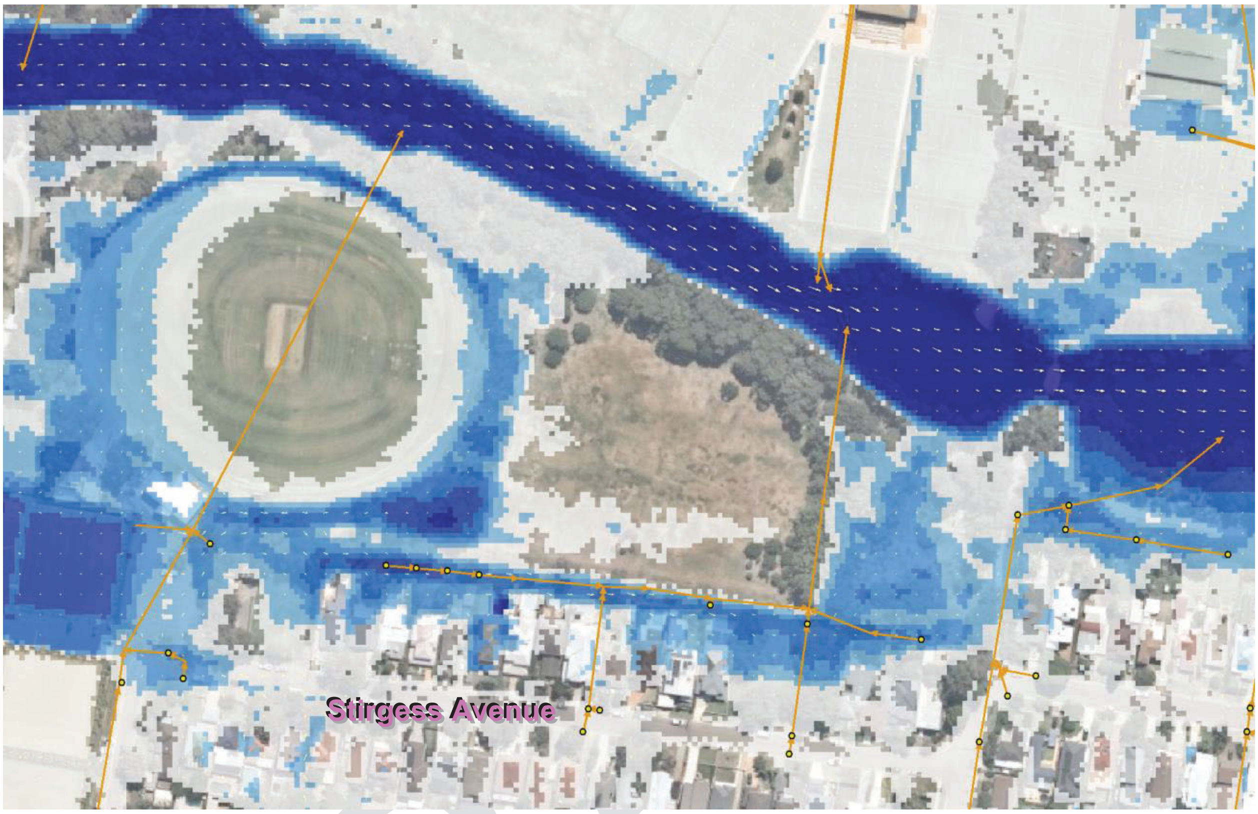
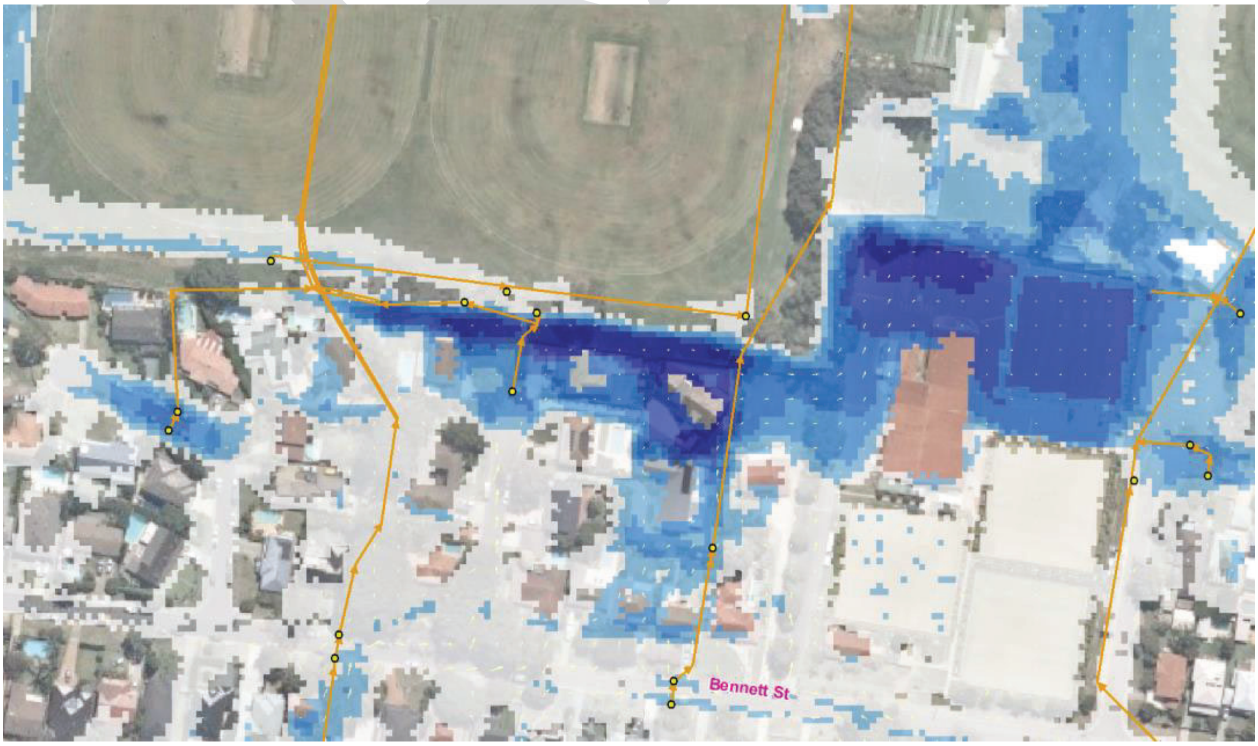


Diagram 15: Low-lying terrain between Bennett Street and Cricket Ovals



There are several playing fields located on reclaimed land south of Greendale Creek. Due to the filling of these areas the playing field surfaces are significantly higher than some of the land to the south, resulting in localised low points where water can accumulate. Modelling indicates the pipes draining these areas are generally full in a 50% AEP event, and flooding will occur in more severe events along the rear of properties on Stirgess Avenue, through the Harbord Bowling and Recreation Club, and other properties that back onto the playing fields (Diagram 14 and Diagram 15).

9.6.8. Harbord Park to Bennett Street

Diagram 16: Overland flow path from Harbord Park to Bennett Street

There is a stormwater drainage line from Harbord Park to Bennett Street, across Brighton Street. The drainage line runs through private property and has capacity ranging from 50% AEP to 10% AEP. In larger storm events exceeding the pipe capacity, overland flow will occur through several properties in this area as indicated on Diagram 16.

9.6.9. Mitchell Road Sag Point near Powells Road

Diagram 17: Mitchell Road Sag Point near Powells Road

There is a confluence of two drainage lines at a sag point on Mitchell Road near Powells Road (Diagram 17). The stormwater pipes downstream of this sag point have a capacity between 50% AEP and 20% AEP, and overland flow will occur in larger events, with depths exceeding 0.5 m in Mitchell Road and properties to the east towards Orchard Road. Overland flow is blocked from exiting the area by buildings and fences between Mitchell Road and Orchard Road / Ada Avenue.