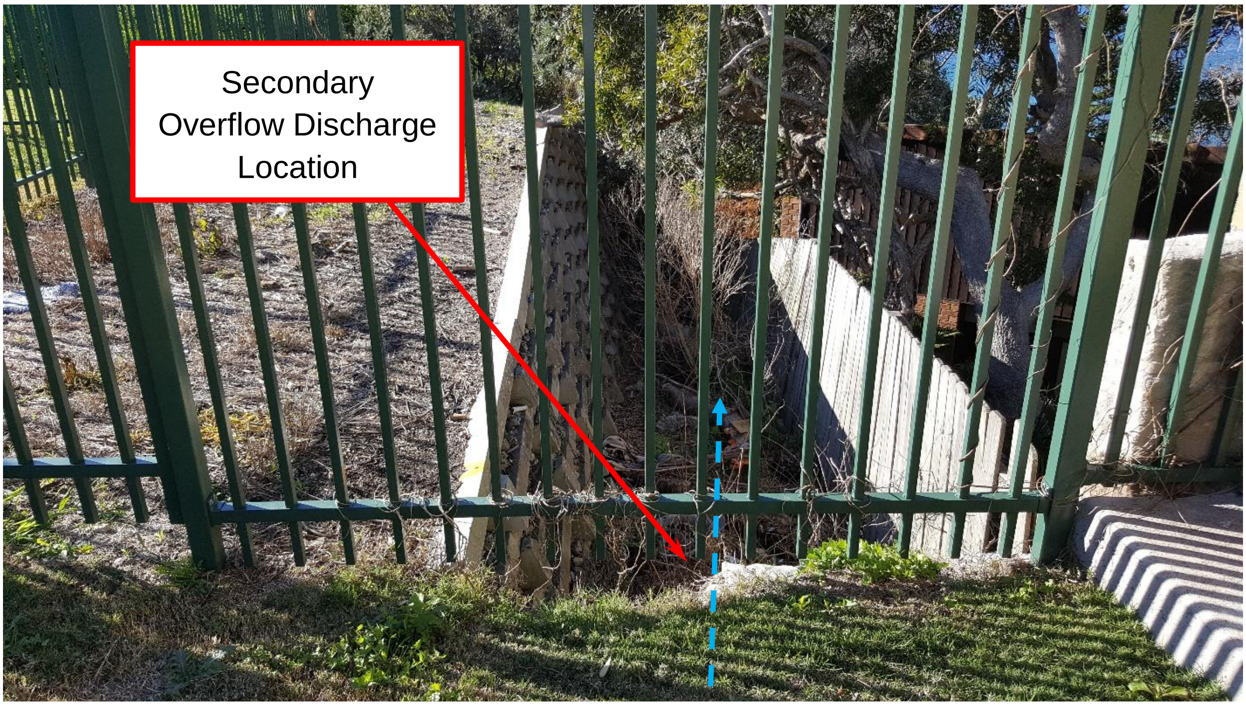

Figure 21 – Eastern view of secondary overflow path discharge point from Site, along southern boundary crib wall

Figure 22 – Western view of secondary overflow path discharge point from Site, along southern boundary crib wall

Figure 23 – Eastern view near secondary overflow path discharge point from Site, along southern boundary crib wall

Figure 24 – Western view from north-western corner Site boundary

4 DRAINS MODELLING

A model of the piped drainage system and corresponding overland flow has been developed using the DRAINS software (Version 2017.07) utilising methods in accordance with the recently released Australian Rainfall & Runoff 2016 (ARR 2016). The critical overland flows related to the Site have than been modelled in HEC-RAS 5 (Version 5.0.3) to determine the Site-specific flood effects.

4.1 Hydrology

4.1.1 Hydrological Model and Parameters

The DRAINS model hydrology has been modelled using Rafts hydrology with initial losses of 0.5 mm and 10 mm and continuing losses of 0 mm and 2 mm for impervious and pervious areas respectively.

The sub catchment areas primarily consisted of residential properties and were typically modelled at 20% pervious area and 80% impervious area, with average catchment slopes generally ranging from 6% to 10% based on LiDAR data.

Design rainfall patterns were developed for the Site's location from Bureau of Meteorology 2016 IFD procedures for the 1% (Annual Exceedance Probability) AEP event covering storm durations from 5 minutes to 3 hours.

4.1.2 Climate Change

The Floodplain Development Manual 2005 requires that Flood Studies and Floodplain Risk Management Studies consider the impacts of Global Climate Change (GCC) on flood behaviour for various Representative Concentration Pathways (RCP). The RCPs are designated as 2.6, 4.5, 6.0 and 8.5, and are named according to radiative forcing values (W m^{-2}) in the year 2100 relative to pre-industrial values.

The ARR recommends the use of Representative Concentration Pathways (RCP) 4.5 and 8.5 (low and high concentrations respectively). RCP 8.5 was modelled out of the two to represent the largest possible impact of climate change. For the Site, the ARR Datahub predicts RCP 8.5 will result in an 18.6% increase in rainfall depth. To address modelling the effects of climate change, a 1.186 multiplier was applied within DRAINS to the BOM IFDs.

4.1.3 Probable Maximum Precipitation

Estimates of Probable Maximum Precipitation (PMP) were made using the Generalised Short Duration Method (GSDM) as described in the Bureau of Meteorology's Bulletin 53 (BOM 2003) which is appropriate for estimating extreme rainfall depths for catchments of this size and storm durations from 15 minutes up to 6 hours.

4.1.4 Critical Storm Durations (1% AEP, 1% AEP + CC and PMP)

Storm 7 from the 10 minute duration was determined as the peak median flowrate (red bar) entering the Site for both the 1% AEP and 1% AEP plus climate change events (Figure 30 and Figure 31), and was adopted for HEC-RAS 2D modelling.

For the PMP event, both the 15 minute and 60 minute durations were identified with similar peak flowrates (Figure 28 and Figure 29). The 60 minute duration was selected for HEC-RAS 2D modelling due to the extended duration of the peak flowrate and larger total volume entering the Site.

4.2 DRAINS Modelling

To determine the maximum overland flowrate approaching the Site on Beach St, a DRAINS model has been developed (Figure 25), based on the catchment characteristics identified in Section 3, and configured to utilise Premium Hydraulic Analysis to assess unsteady overland flows approaching the Site.

No allowance has been made in these calculations for properties that may have Onsite Detention (OSD) tanks or Rainwater Tanks within the catchment. This provides a conservative assessment and it is possible that flows at the Site could be less.

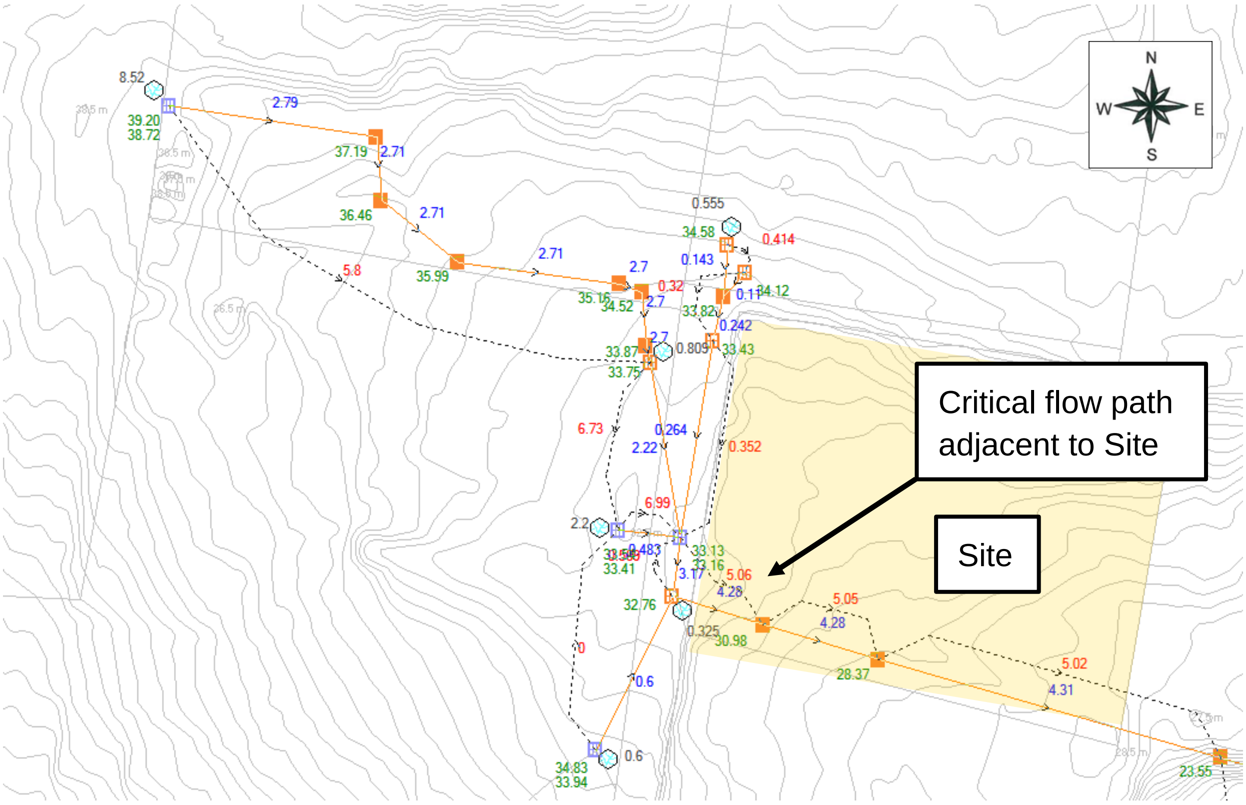

Figure 25 – Configuration of DRAINS model

4.3 Model Geometry

Pit and Pipe Data

Council supplied GIS files of all pits and pipes for the catchment. The data included the size and type of each pipe, and the type of pit (grated, junction). Pit surface levels were interpolated from the LiDAR data and a minimum cover of 600 mm was adopted for the pipes to set invert levels. A 50% pit blockage factor was applied to sag pits.

Pit loss coefficients were assigned initial values and then repeatedly revised in accordance with the Missouri Charts utilising the built-in DRAINS ‘Revise Pit Loss Coefficients’ procedure until values converged.

Overflow Routes

Overflow routes were typically determined via a combination of data sources, where available, including LiDAR data, contours generated from LiDAR data, and the during Site investigations. Overflow routes were configured with Premium Hydraulic parameters to best assess each overflow.

4.4 Pre-Developed Model Results

Following are the results from the DRAINS Premium Hydraulic Analysis.

4.4.1 Overland flows entering the Site from Beach Street

Pre-developed critical overland flows have been modelled entering the Site from Beach Street. Figure 32 shows a 1% AEP peak median flowrate of 5.06 m^3/sec, Figure 33 shows a 1% AEP + Climate Change peak median flowrate of 5.93 m^3/sec, and Figure 34 shows the Probable Maximum Flood (PMF) flowrate of 14.9 m^3/sec. The critical duration hydrographs of these results (Figure 26, Figure 27 and Figure 29) were utilised for pre-developed conditions HEC-RAS 2D modelling.

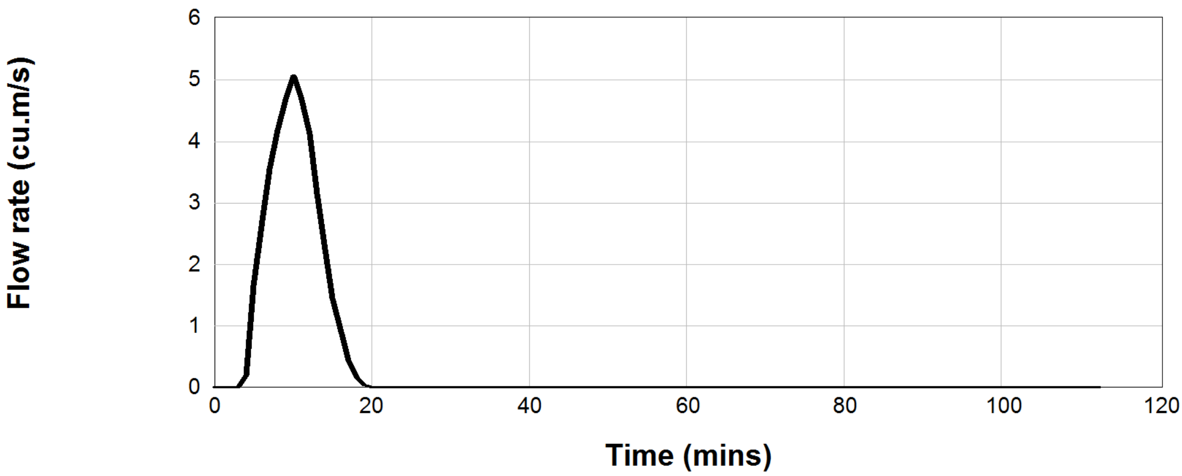

Figure 26 – Pre-developed 1% AEP, 10 minute duration, storm 7 critical hydrograph entering the Site

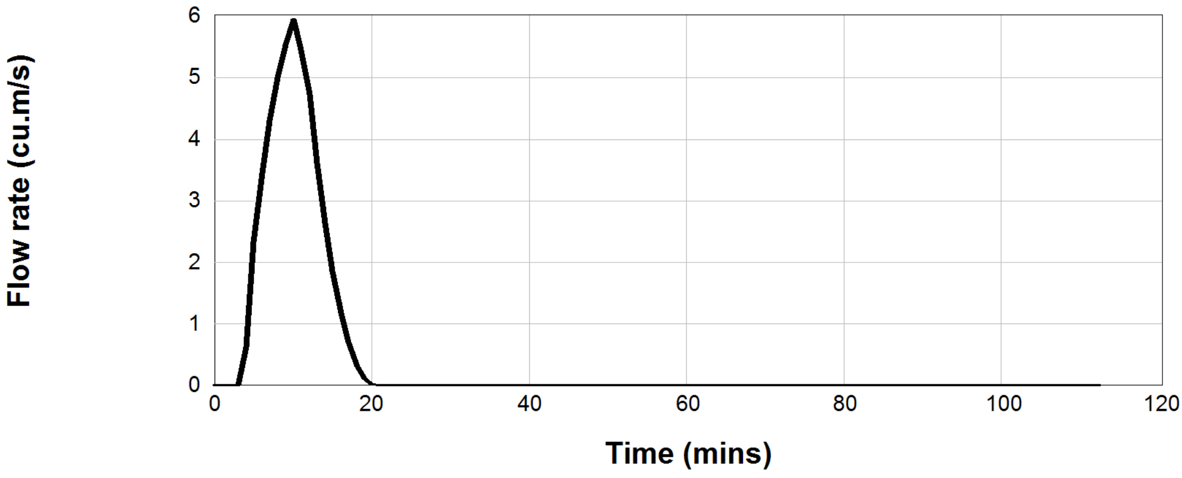

Figure 27 – Pre-developed 1% AEP + Climate Change, 10 minute duration, storm 7 critical hydrograph entering the Site

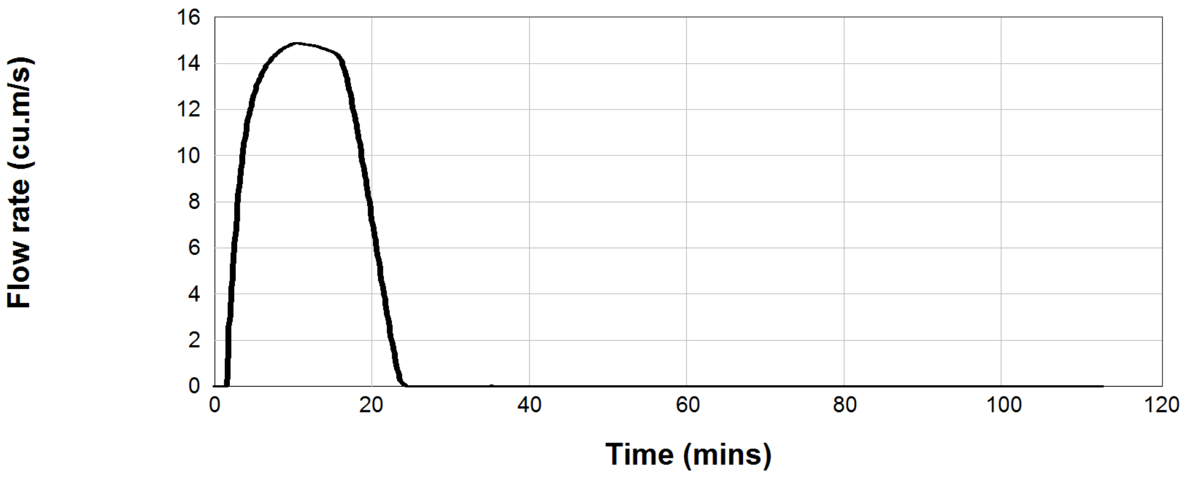

Figure 28 – Pre-developed PMF, 15 minute duration, critical hydrograph entering the Site

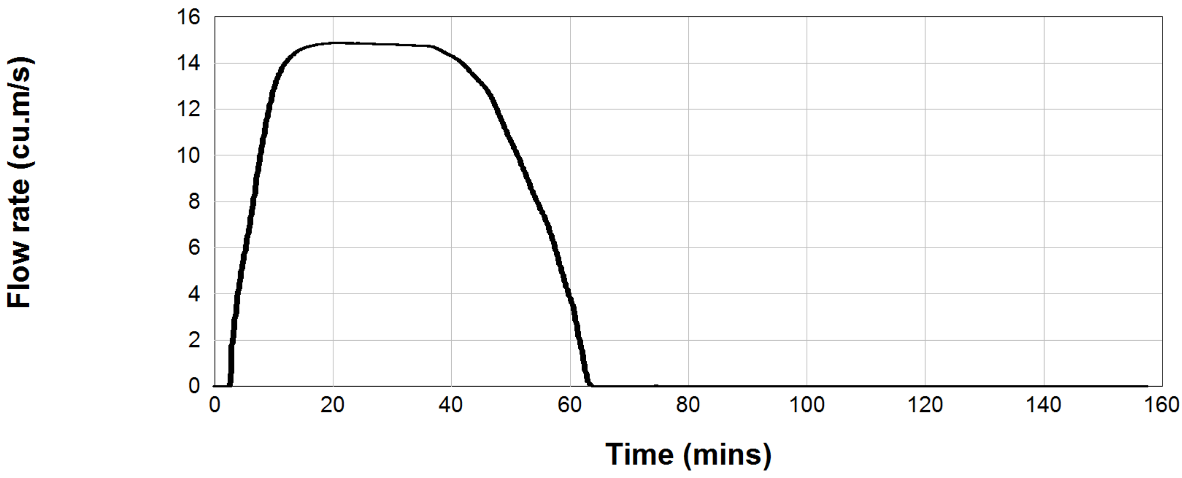

Figure 29 – Pre-developed PMF, 60 minute duration, critical hydrograph entering the Site

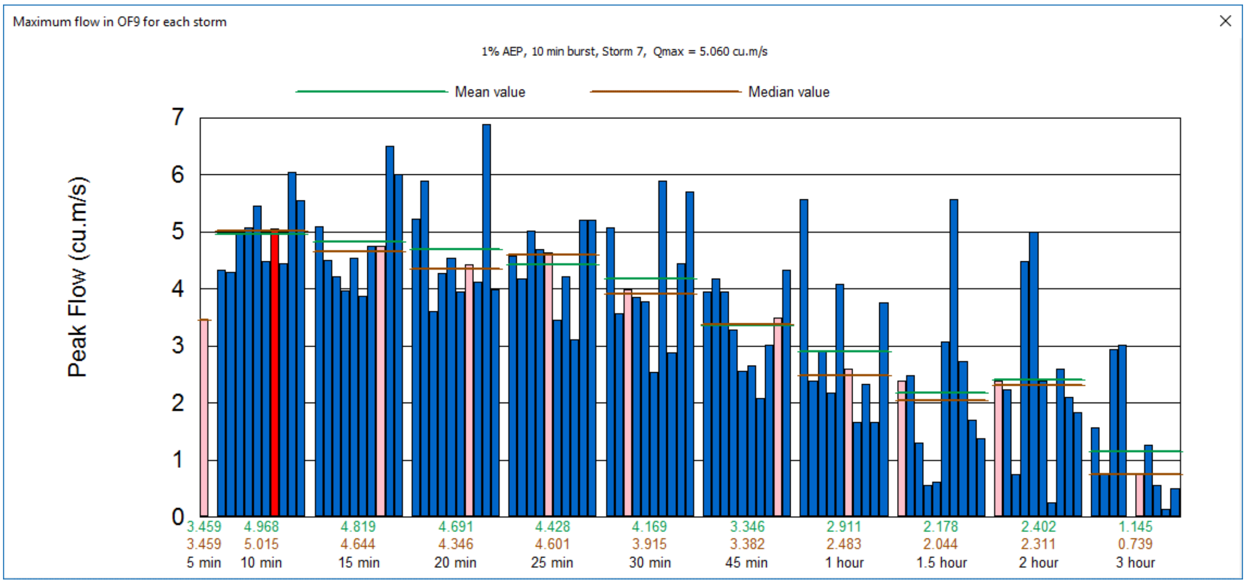

Figure 30 – Pre-developed 1% AEP flowrate of peak median storm (red) entering the Site

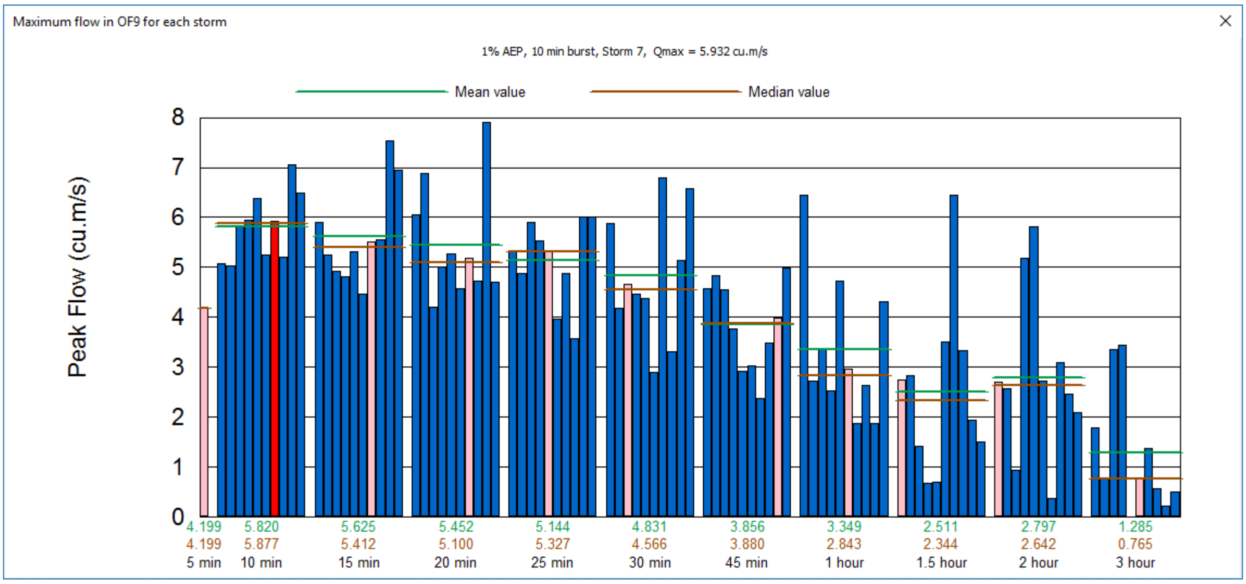

Figure 31 – Pre-developed 1% AEP + Climate Change flowrate of peak median storm (red) entering the Site

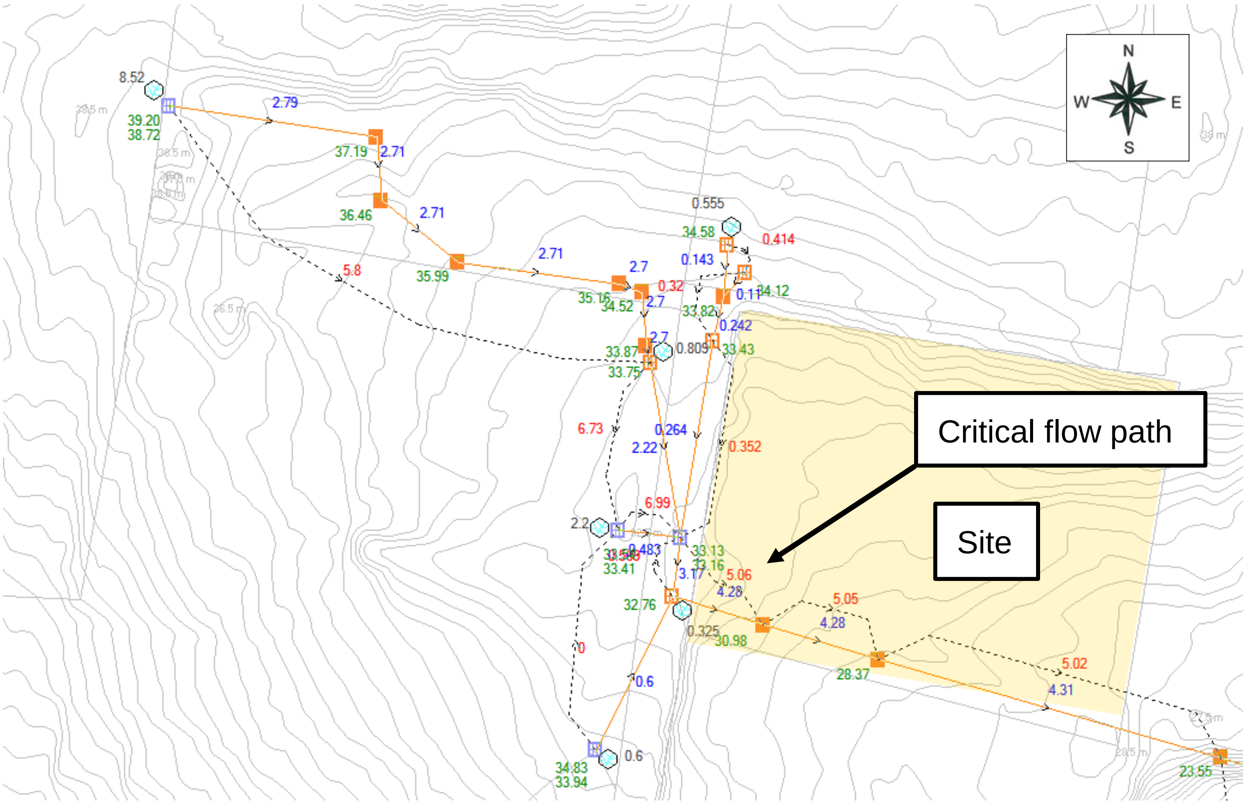

Figure 32 – Pre-developed DRAINS Model 1% AEP results

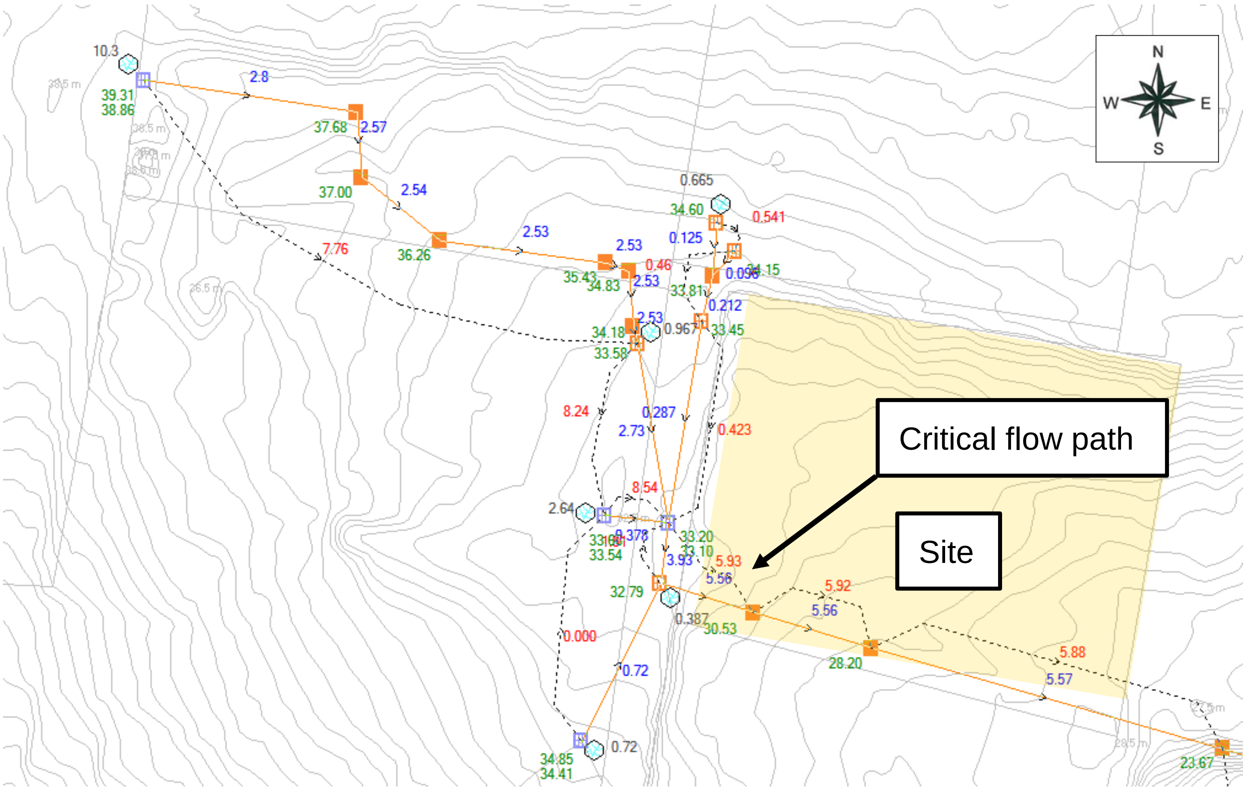

Figure 33 – Pre-developed DRAINS Model 1% AEP results + climate change

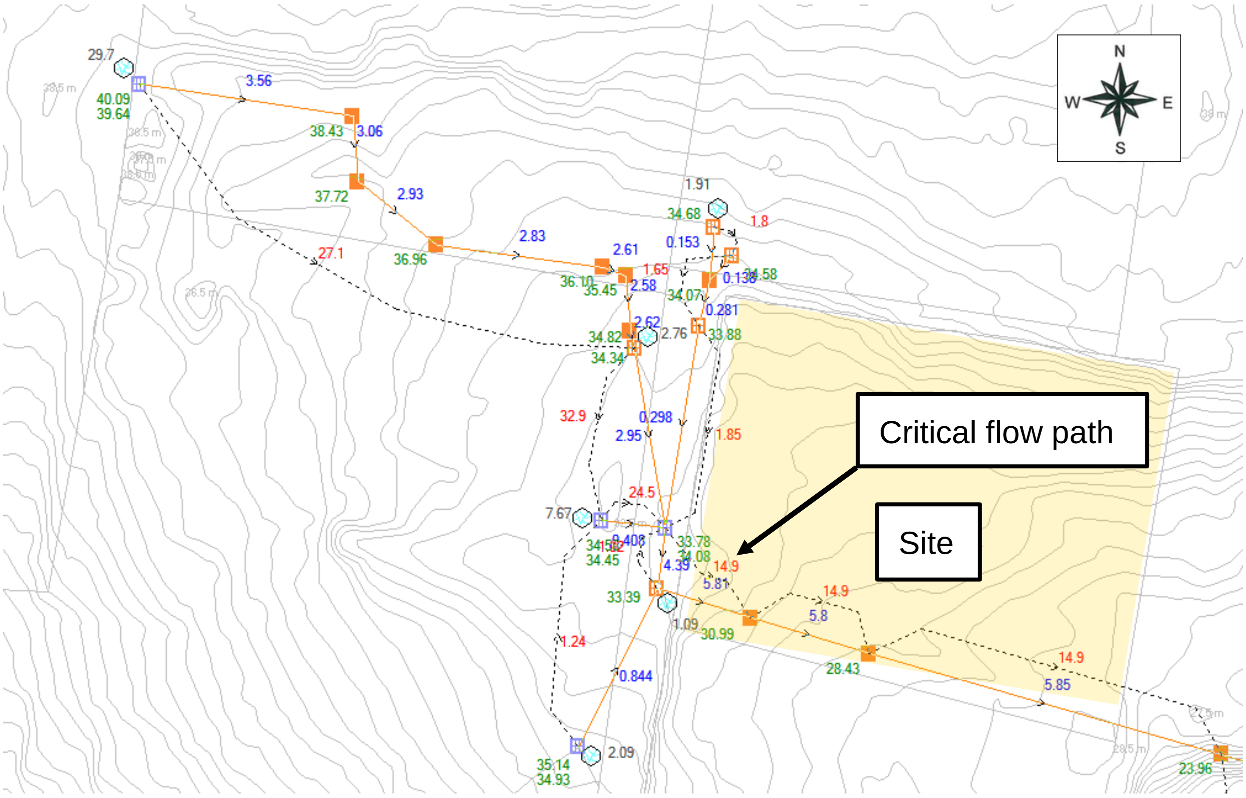

Figure 34 – Pre-developed DRAINS Model PMF Results

4.5 Post-Developed Model Results

The flow path (gate) entering the Site was widened to 6.5 m to accommodate large vehicles and pedestrians entering the Site. This modification resulted in a slight increase to the flowrate entering the Site, while slightly reducing ponding levels in Beach Street.

Following iterative 2D flood modelling utilising the full overland flowrate and the proposed development, it was determined that conditions would not be safe for pedestrians and vehicles on the driveway along the southern boundary (Figure 35) due to high velocities, and the entry to the basement car park would be at risk of flooding.

In order to provide safer overland flow conditions, an inlet structure is required to capture overland flow entering the Site and reduce the overland flowrate to no greater than 2.00 m^3/sec by the parking zone. The method, location, number, and type of inlet structure/s is flexible. This could be achieved via a single large grated inlet structure near the gate, or through several inlet structures commencing in the street – possibly through upgrading Council's existing system. The final specification of the inlet, culvert and outlet structures design will need to be completed by others.

For the purposes of producing a hydrograph for HEC-RAS 2D modelling, an oversized grate structure was positioned near the Site entrance gate with twin 825 mm dia. pipes draining to an outlet structure with dissipation, located in the grass area downstream of the basement carpark entrance, adjacent to the staircase (Figure 35).

Following are the results from the DRAINS Premium Hydraulic Analysis.

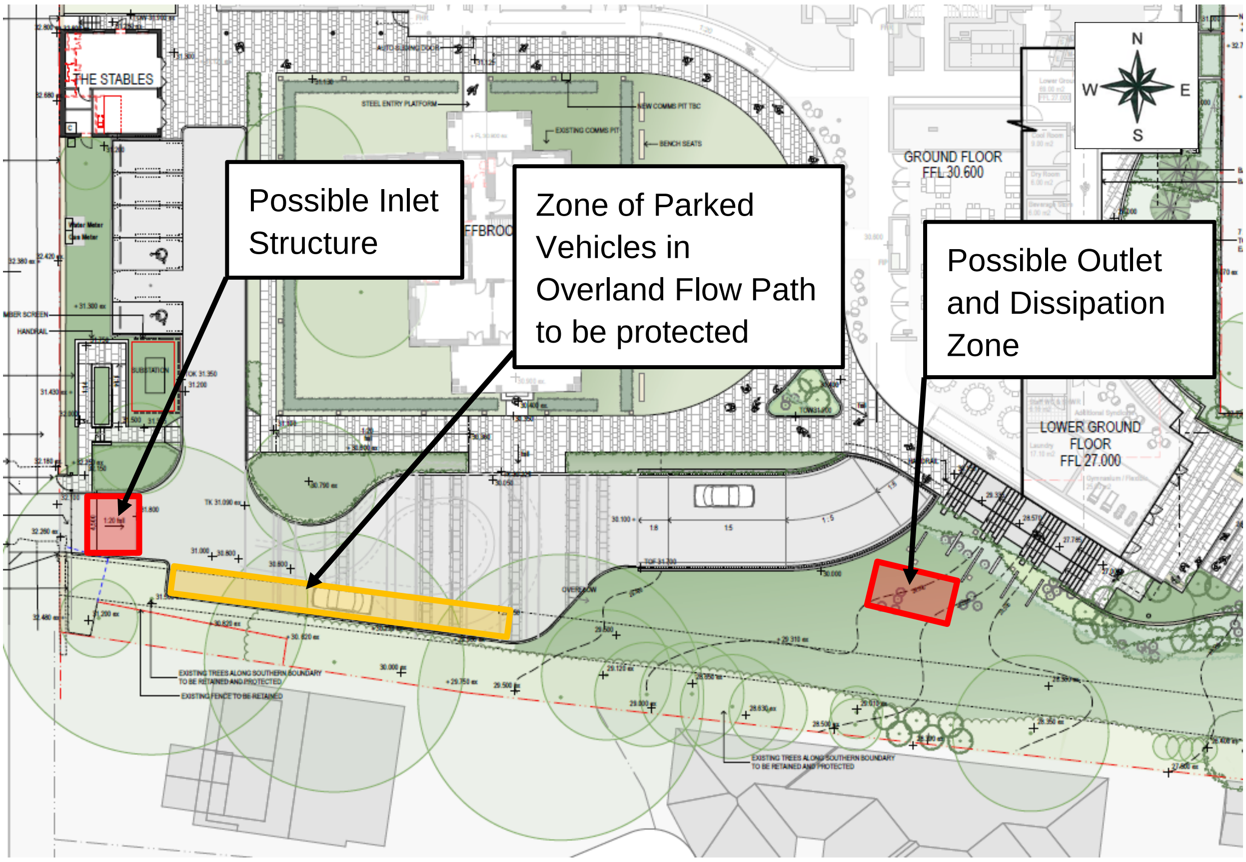

Figure 35 – Possible locations of required inlet and outlet structures with dissipation

4.5.1 Overland flows entering the Site from Beach Street

Post-developed critical overland flows have been modelled entering the Site. Figure 36 shows a 1% AEP peak median flowrate of 2.00 m^3/sec, Figure 37 shows a 1% AEP + Climate Change peak median flowrate of 3.38 m^3/sec, and Figure 39 shows the PMF flowrate of 11.9 m^3/sec. The critical duration hydrographs of these results (Figure 36, Figure 37 and Figure 39) were utilised for pre-developed conditions HEC-RAS 2D modelling.

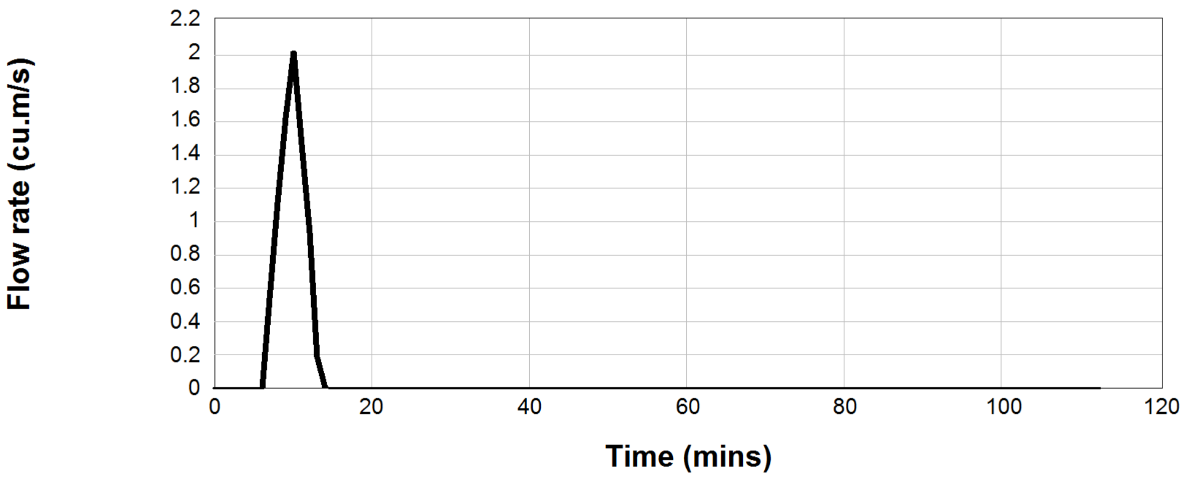

Figure 36 – Post-developed 1% AEP, 10 minute duration, storm 7 critical hydrograph entering the Site

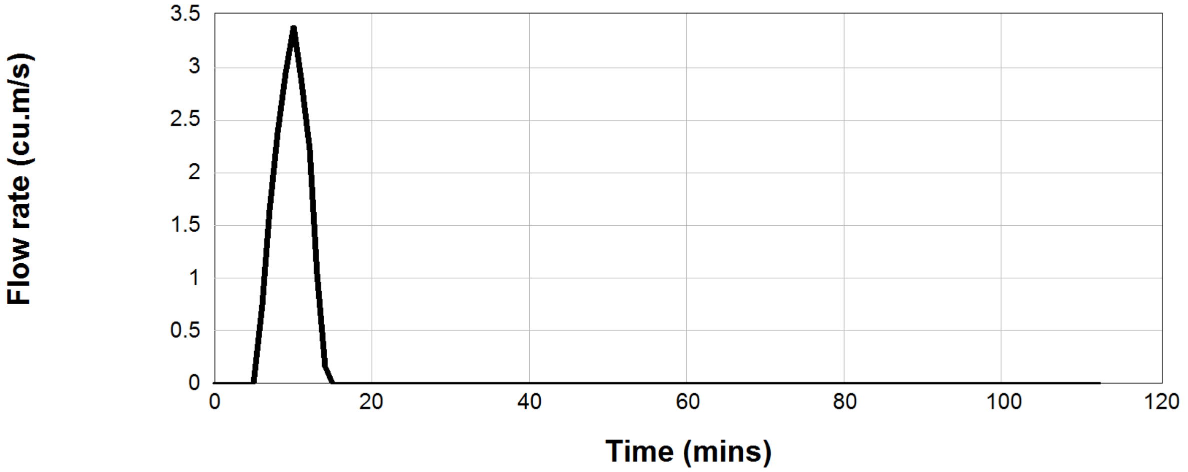

Figure 37 – Post-developed 1% AEP + Climate Change, 10 minute duration, storm 7 critical hydrograph entering the Site

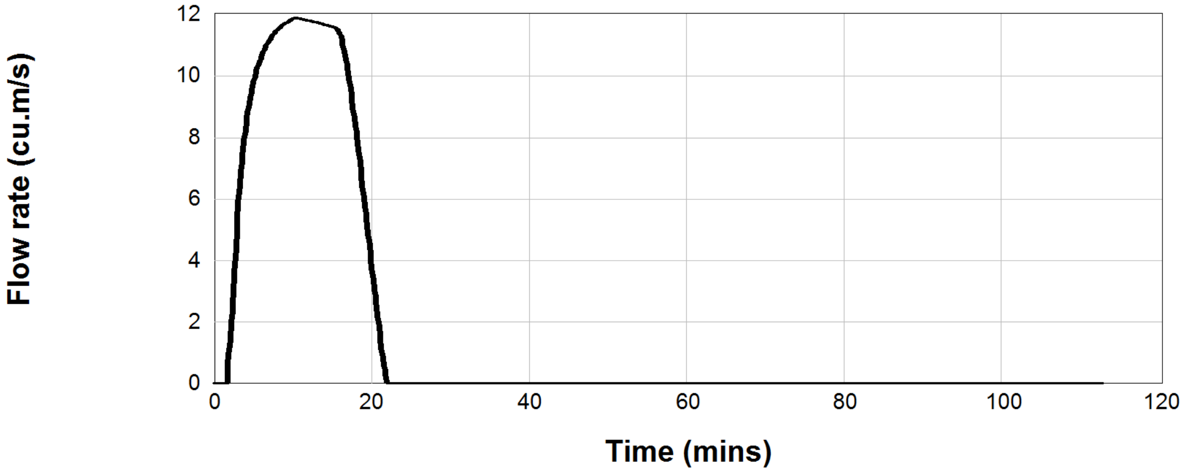

Figure 38 – Post-developed PMF, 15 minute duration, critical hydrograph entering the Site

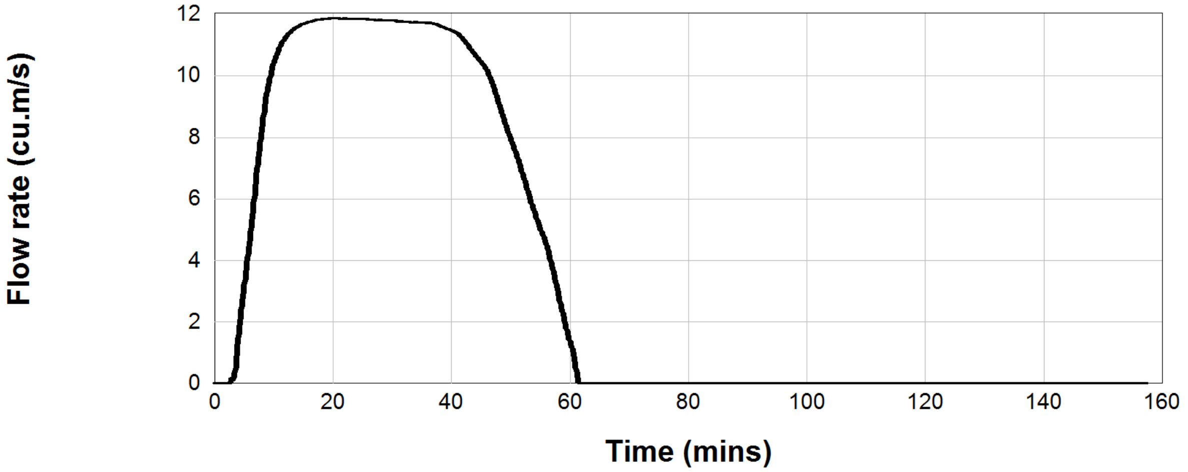

Figure 39 – Post-developed PMF, 60 minute duration, critical hydrograph entering the Site

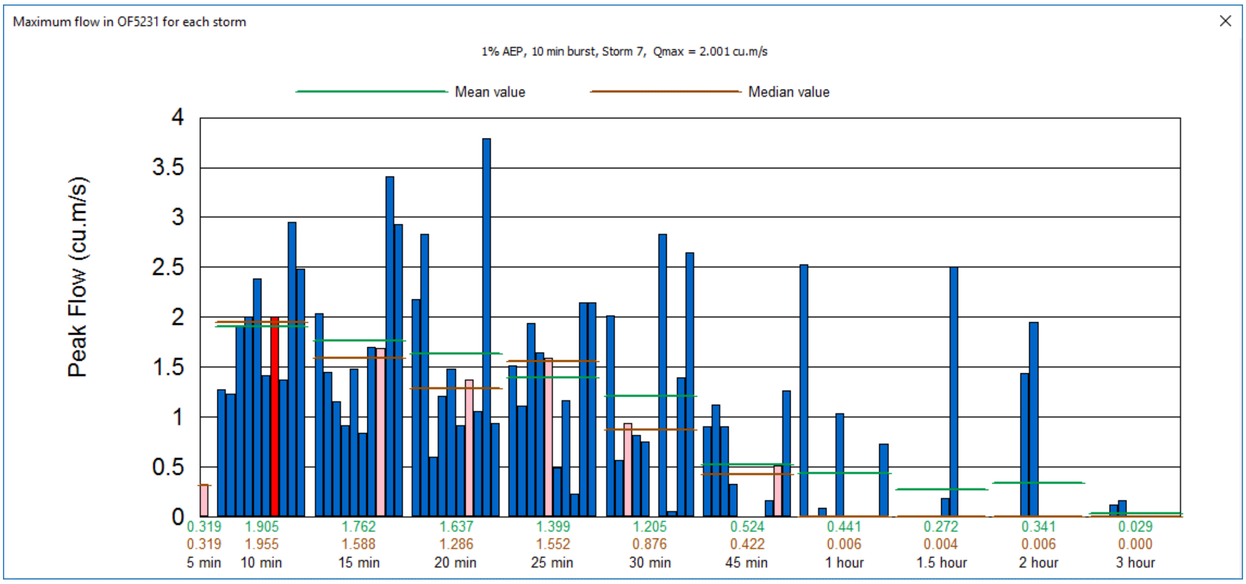

Figure 40 – Post-developed 1% AEP flowrate of peak median storm (red) entering the Site

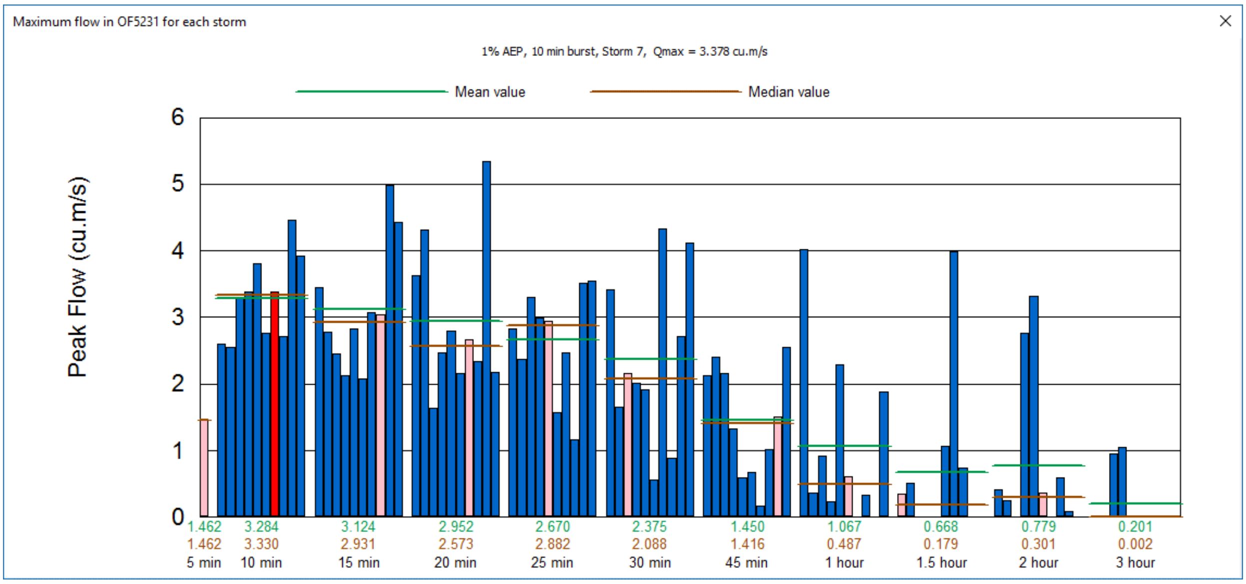

Figure 41 – Post-developed 1% AEP + Climate Change flowrate of peak median storm (red) entering the Site

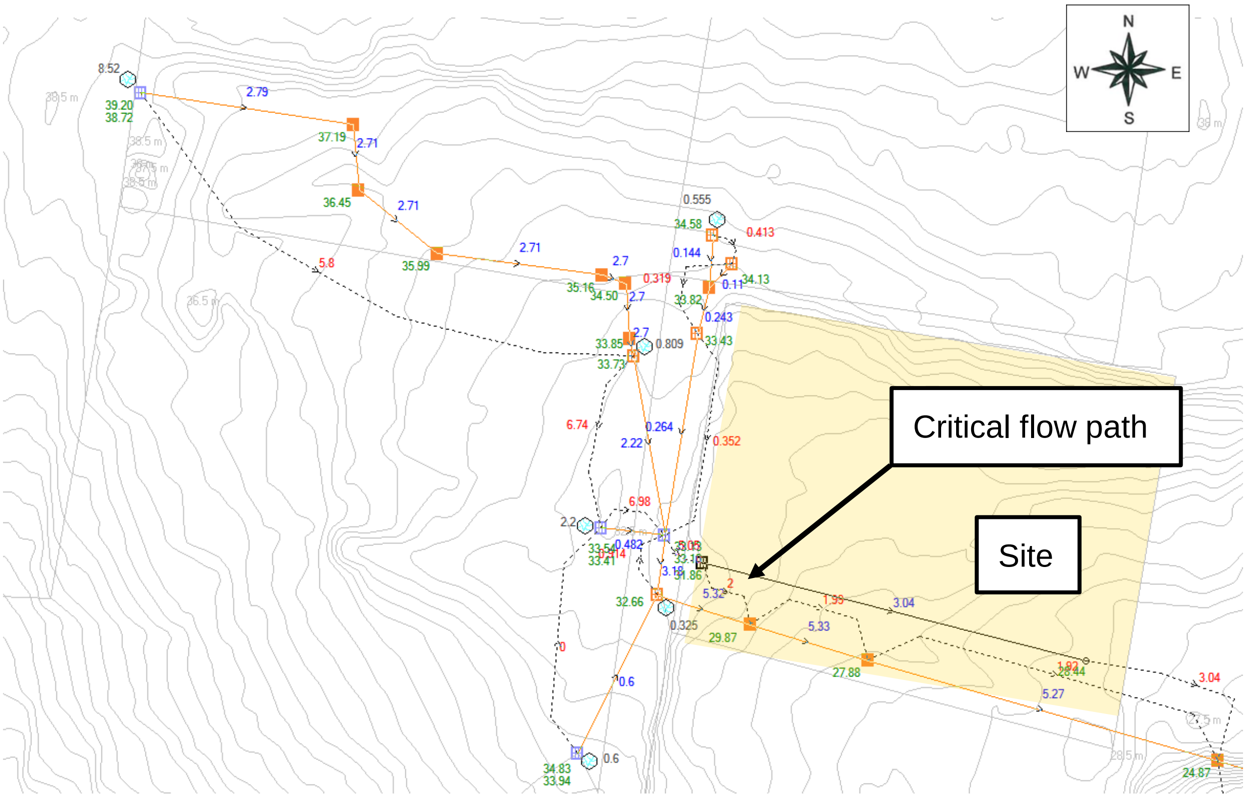

Figure 42 – Post-developed DRAINS Model 1% AEP results

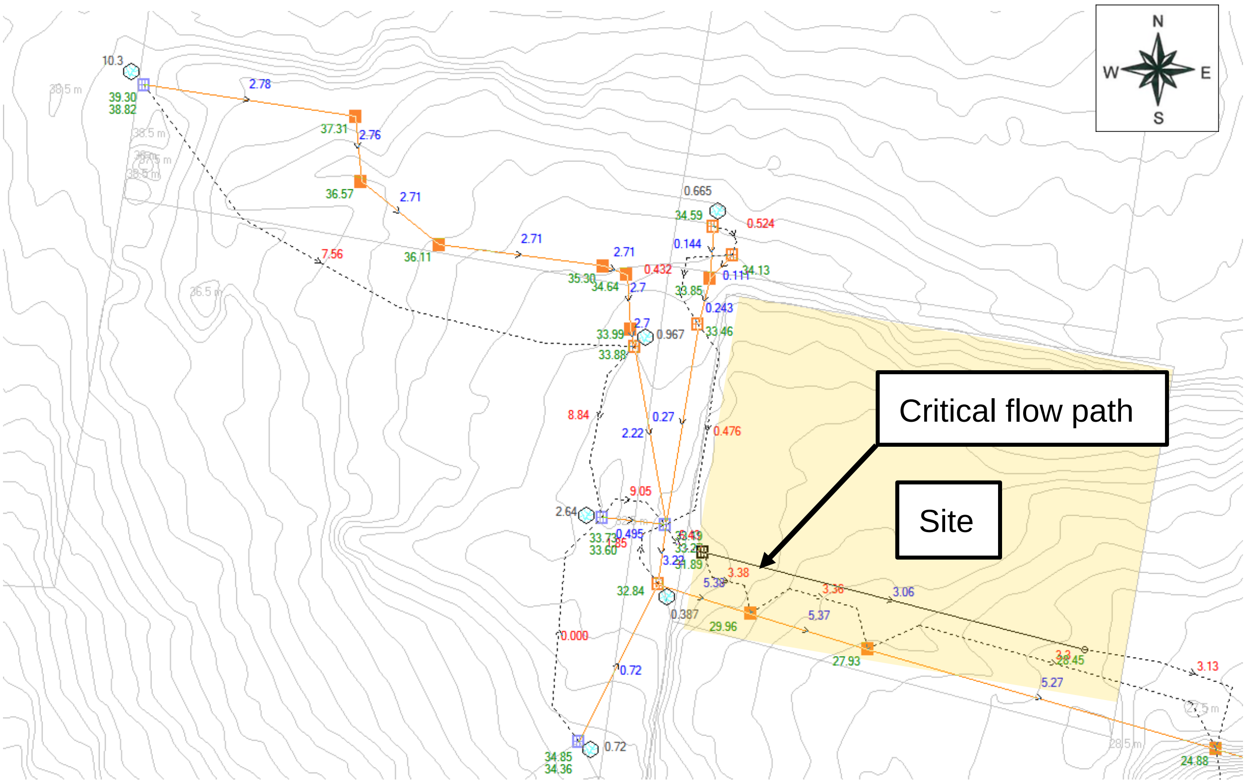

Figure 43 – Post-developed DRAINS Model 1% AEP results + climate change

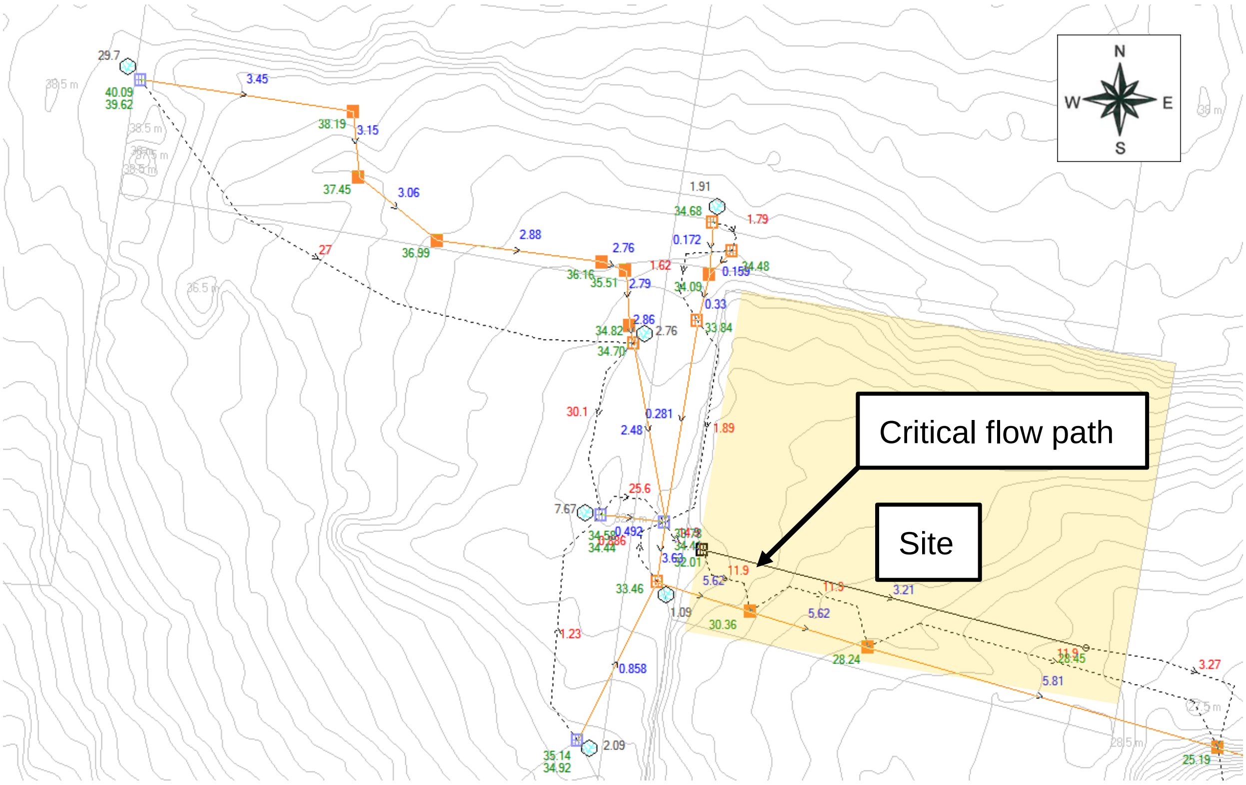

Figure 44 – Post-developed DRAINS Model PMF Results

5 HEC-RAS 2D MODELLING

5.1 HEC-RAS Model

For analysis of Site specific flood characteristics, a HEC-RAS 2D model was developed from NSW LPI LiDAR information and Site specific survey (Figure 2 & Figure 3) for the pre-developed conditions. The eastern downstream end of the Site contained dense vegetation which is known to affect the levels of LiDAR data. The affected areas were modified to match the detailed survey ground levels.

For the developed conditions, several model iterations of the proposed redevelopment were required due to the high flood risks associated with the entrance to basement car park and freeboards. The scenario shown in Figure 5 is generally the final model adopted, although with further modifications incorporated around and downstream of the lower ground floor to achieve a better compliant flood result. With a proposed lower ground floor level of 27.00 m AHD, Figure 5 proposes all flush entry points into the building, which results in flat terrain and is generally not compatible with flooding.

A flush entrance (27.00 m AHD) into the northern most lower ground floor entrance has been modelled, as this area is undercover and is unlikely to be flood affected this far upstream of the primary flow path, however, south of this location a minimum fall of 0.5% slope was modelled. This results in a maximum level of 26.875 m AHD near the south eastern lower ground floor entrances, which is approximately a 125 mm step into the building. A minimum fall of 1% slope was then modelled away from this location in a south easterly direction resulting in further levels downstream needing to be adjusted.

A dynamic flexible mesh (Figure 46 & Figure 46) with cells no larger than 1 m by 1 m was developed over the Site from Beach Street through to the bush land to be retained at the eastern lower end of the Site. Break lines were entered around structures to model obstructions to improve grid boundary weir flow relationships.

The unsteady flow critical input hydrograph data was determined from the DRAINS model as outlined in Section 4.4.1 and was applied as inflow to Beach Street. Lower boundary conditions were set to normal depth. Unsteady flow computation was set to 0.1 second intervals with the Diffusion Wave equation set and a mixed flow regime.

HEC-RAS 'Landcover' shapefiles were prepared in GIS for the pre and post developed conditions to model the spatial distribution of surface roughness. As flows were expected to be generally deeper than 100 mm, the Manning's 'n' values shown in Table 1 were applied to the shapefiles.

Buildings located in the flood path were elevated above the flood levels to prevent any overestimation of flood storage volumes, while short walls and other flood diversion structures that could be overtopped during flooding were raised to surveyed AHD levels (Figure 46).

Table 1 – Land Use Manning's 'n' values

Land Use	Manning's 'n' Value
Grass	0.030
Road Pavement	0.015
Concrete	0.015
Garden beds	0.050
Dense vegetation (including hedges)	0.100

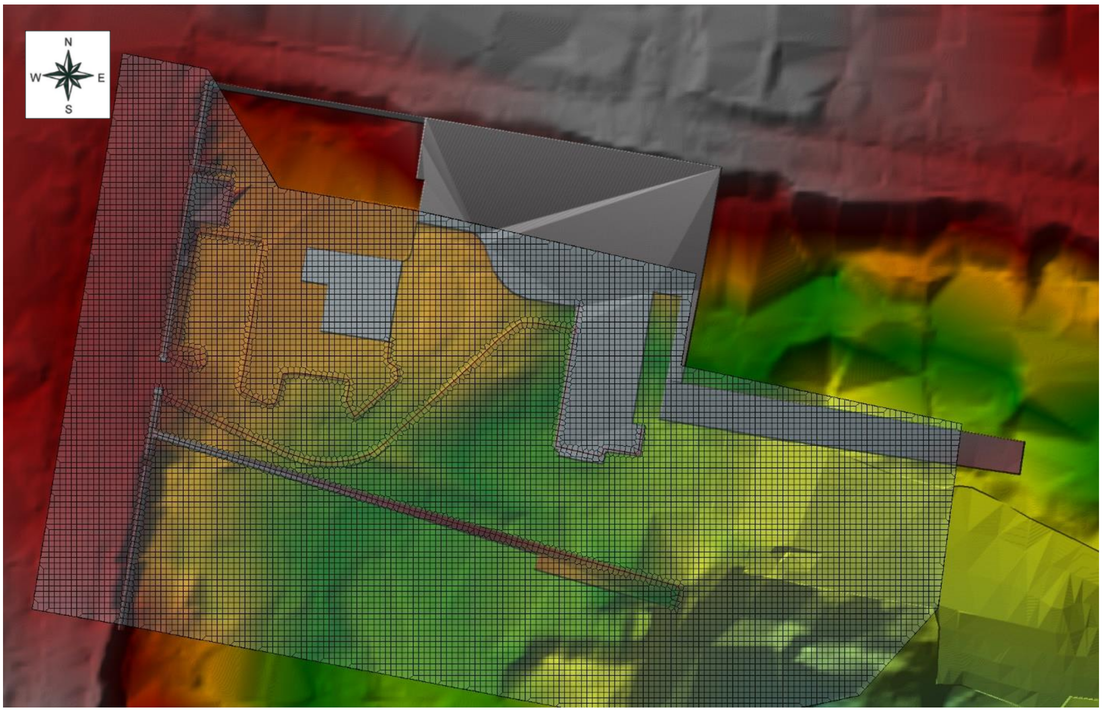

Figure 45 – Pre Developed flexible mesh and break lines

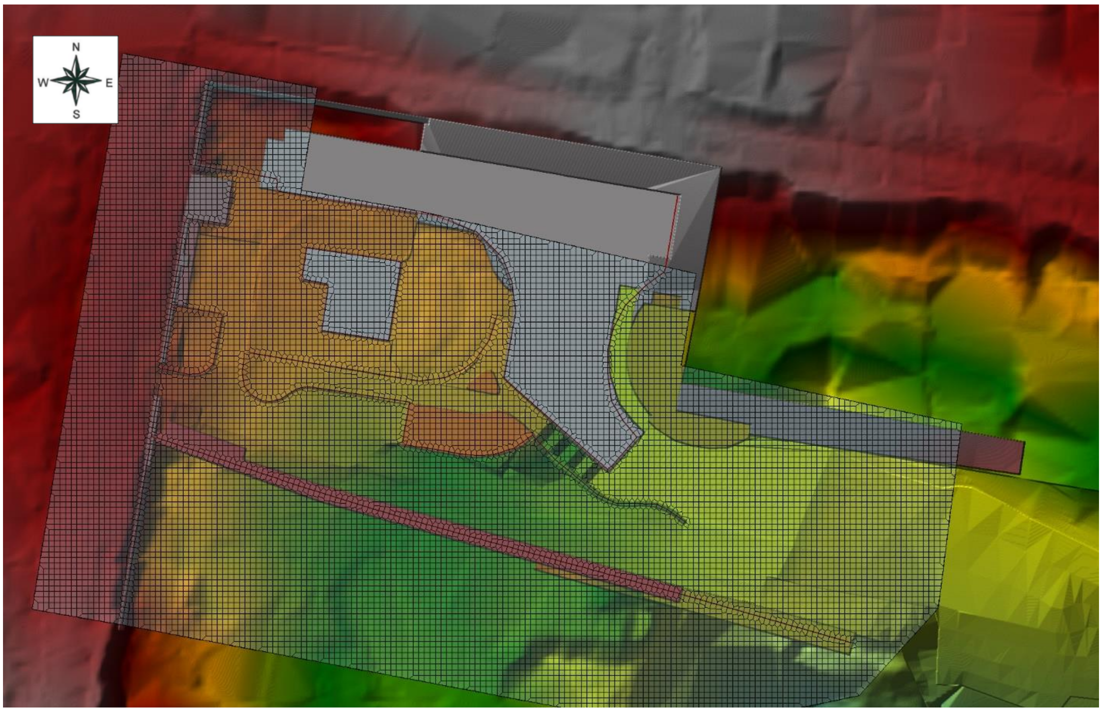

Figure 46 – Post Developed flexible mesh and break lines

6 FLOOD MODELLING RESULTS

6.1 Review of Pre-Developed Flood Results

6.1.1 Pre-Developed Peak Flood Depths 1% AEP

Pre-developed flood affectation in terms of depths, extents and level contours is shown for the 1% AEP design event on Figure_A 1. Peak flood depths reach 430 mm in Beach Street near the Site entrance due to ponding as flow is restricted through the Site gate entry.

Upon entering the Site, flows generally fall east down the driveway towards the weir and primary overland flow path, while also topping the kerb into the secondary overland flow path. Due to a limited hump and grading of the driveway entrance, flows will also break out toward the northern car park sag point (Figure 14) and eventually overtop the eastern kerb resulting in shallow flow around the Cliffbrook House typically less than 100 mm in depth.

Peak flood depths reach up to 650 mm along the southern boundary adjacent to the existing property before meandering around a mound and tree then discharging into a gully adjacent to the crib wall.

6.1.2 Pre-Developed Peak Flood Velocities 1% AEP

Peak velocities shown in Figure_A 3 are characterised by the depth of ponding in Beach Street and the steep gradient from the Site entry and down the driveway. Through the entrance, velocities exceed 6.5 m/s and typically exceed 5 m/s along the southern kerb until the weir. Peak velocities along the footpath between Cliffbrook House and the car park reach up to 4.5 m/s.

Within the primary overland flow path peak velocities typically range from 2.0 to 2.5 m/s due to the wide grassed nature of the path, however, velocities reach 3.5 m/s when converging with the secondary overland flow path downstream near the mound & tree before reaching the vegetation area.

Peak velocities in the secondary overland flow path typically range from 2.0 to 3.0 m/s in the grassed area, and are typically less than 1.0 m/s along the southern boundary edge in the vegetated area.

6.1.3 Pre-Developed Peak Flood Velocity Depth Product 1% AEP

The peak flood velocity depth product (VxD) is shown in Figure_A 5. Due to the high velocity and depth of flood water discharging through the entrance, the VxD exceeds 0.8 m^2/s. VxD is reduced along the driveway and southern car parking spaces, however, VxD still remains between 0.5 m^2/s and 0.85 m^2/s.

Within the primary overland flow path peak VxD typically ranges from 0.33 m^2/s to 0.55 m^2/s, however, VxD appears to be higher when converging with the secondary overland flow path downstream near the mound & tree before reaching the vegetation area, reaching a VxD of 0.7 m^2/s.

Peak VxD is also quite high in the secondary overland flow path reaching up to 0.98 m^2/s in the grassed area, while typically remaining under 0.6 m^2/s along the southern boundary edge in the vegetated area.

6.1.4 Pre-Developed Peak Flood Depths PMF

Flood affectation in terms of depths, extents and level contours is shown for the PMF design event on Figure_A 2. Flooded extents typically replicate the flood paths of the 1% AEP event, but to a greater extent. Peak depths reach 620 mm in Beach Street, up to 200 mm around Cliffbrook House, up to 690 mm in the primary overland flow path and up to 780 mm along the southern boundary in the secondary overland flow path. Due to the higher depths in the northern car park, water is likely to start topping the northern kerb and enter The Stables. This increased depths also results in flows passing around the northern side of Cliffbrook House. It is possible that flood water may enter the Cliffbrook House and The Stables building during a PMF event.

6.1.5 Pre-Developed Peak Flood Velocities PMF

Peak velocities for the PMF event are shown in Figure_A 4. Through the entrance, peak velocities exceed 10.0 m/s and typically exceed 6.0 m/s along the southern kerb until the weir. Peak velocities along the footpath between Cliffbrook House and the car park exceed 6.0 m/s.

Within the primary overland flow path peak velocities typically range from 3.0 to 4.0 m/s, however, velocities reach 5.0 m/s when converging with the secondary overland flow path downstream near the mound & tree before reaching the vegetation area.

Peak velocities in the secondary overland flow path typically range from 3.0 to 4.5 m/s in the grassed area, and are typically less than 1.5 m/s along the southern boundary edge in the vegetated area.

6.1.6 Pre-Developed Peak Flood Velocity Depth Product PMF

Peak VxD for the PMF event is shown in Figure_A 6. Values are amplified compared to the 1% AEP event due to the increase in both depth and velocity. The average peak VxD across the Site entrance is 2.5 m^2/s, and VxD typically exceeds 1.0 along the remainder of the driveway and southern parking spaces, with peak values of VxD reaching 1.9 m^2/s. Both the primary and secondary overland flow paths typically exceed 1.0 m^2/s, with peak VxD values of 1.6 m^2/s and 2.0 m^2/s respectively.

6.2 Review of Post-Developed Site Specific Flood Results

The proposed redevelopment scenario has been modelled by constructing a detailed surface terrain model of features shown on the proposed plans provided in Figure 5 and Figure 6, incorporating the required alterations listed in Section 5.1. modelled features included items such as the proposed building, driveway, kerbs, pedestrian footpaths and staircase, basement car park entrance, and the remodelled overland flow paths. To ensure minimal adverse impacts on surrounding properties and existing trees, existing levels were throughout the Site were generally retained.

Due to HEC-RAS' inability to model inlet and outlet pit structures during 2D modelling, a combination of three flood maps were stitched together to form a composite map (Figure 47), representing the change in overland flow conditions.

Flood maps containing the full peak flow hydrographs were utilised for mapping the results upstream of the proposed grated inlet structure, and downstream of the proposed outlet structure. The reduced peak flow hydrograph representing the remaining overland flow, was utilised to represent reduced flood conditions in-between the two structures.

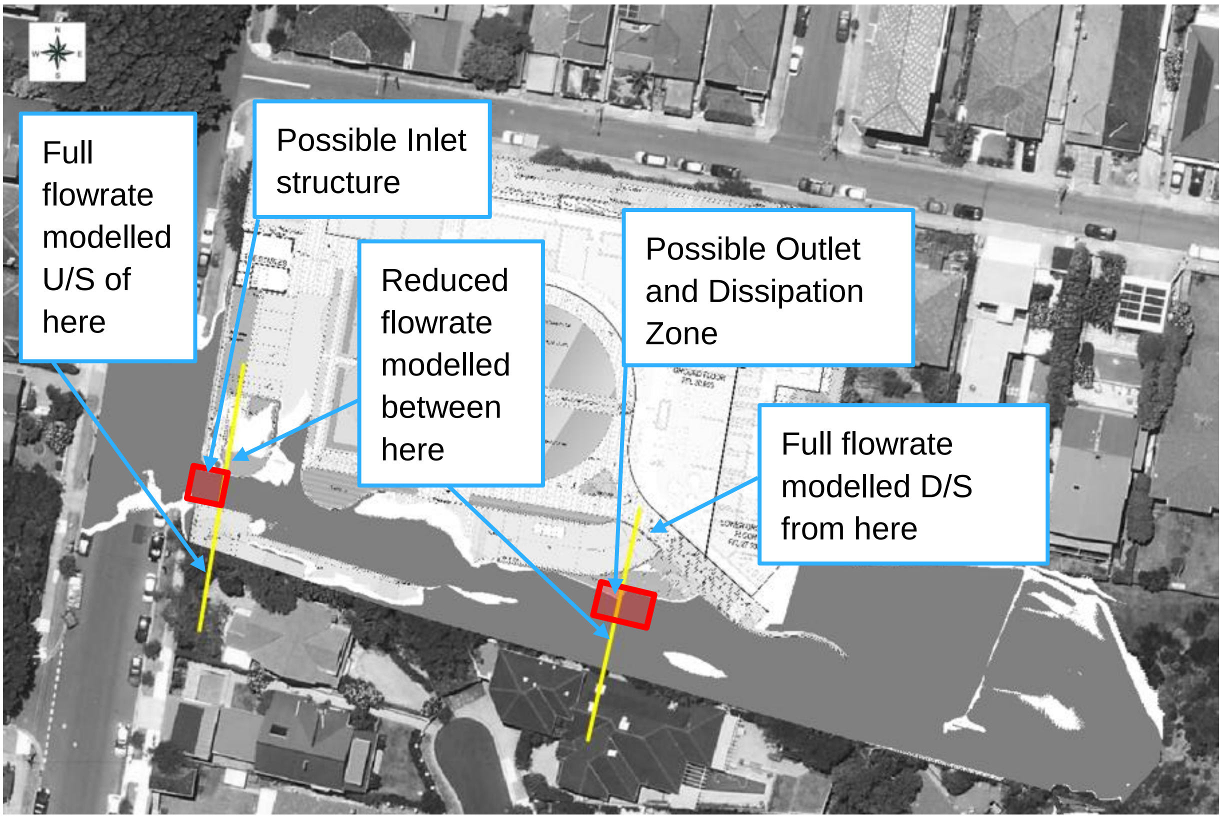

Figure 47 – Allocation of flow rates for post-development composite flood maps

6.2.1 Post-Developed Peak Flood Depths 1% AEP

Post-developed flood affectation in terms of depths, extents and level contours is shown for the 1% AEP design event on Figure_A 9. Peak flood depths have reduced slightly in Beach Street due to the slightly wider entrance being proposed (Figure_A 21).

Upon entering the Site, after water is captured in the proposed grated inlet structure, flood depths have significantly reduced to under 100 mm along the southern car parking spaces. Flood water no longer flows into the northern car parking area, nor does it overtop the kerb and flow around Cliffbrook House (Figure_A 21).

Due to the remodelling of terrain downstream of the proposed outlet and dissipation structure, flood depths have typically reduced to depths of 50 mm to 100 mm. Flows no longer pass from the secondary overland flow path into the adjacent private property due to the crib wall being removed and a new wall and fence being extended along the southern boundary (Figure_A 21). Flow is now more evenly dispersed across the lower section of the Site before being discharged directly towards Gordons Bay. Even though water enters the private property in the existing condition, altering the wall could lead to water entering the private property at a different location and result in new unintended scouring and water flood damages. It is recommended that the new wall be extended at least 300 mm above the ground level to aid in keeping water within the Site and away from the adjacent private properties.

The highest flood level within 6 metres of the entrance to the basement car park is 30.00 m AHD. The peak level at the weir adjacent to the entrance is 29.80 m AHD.

6.2.2 Post-Developed Peak Flood Velocities 1% AEP

Post-developed peak velocities are shown in Figure_A 12. While velocities remain similarly high to pre-developed conditions through the Site entrance (exceeding 6.5 m/s m/s), after being captured in the proposed grated inlet structure, velocities typically reduce to less than 3 m/s along the southern car parking spaces, except for a small section that reaches up to 3.1 m/s.

Within the primary overland flow path, peak velocities may reach up to 4.0 m/s near the outlet structure, which is why a dissipation structure is important at this location. Typically, however, the rest of the overland flow route remains between 2.0 to 3.0 m/s.

Peak velocities in the secondary overland flow path have been reduced from existing conditions and typically range from 1.0 to 2.5 m/s in the grassed area, and are typically less than 1.0 m/s along the southern boundary edge in the vegetated area.

6.2.3 Post-Developed Peak Flood Velocity Depth Product 1% AEP

Peak VxD for the post-developed 1% AEP event is shown in Figure_A 15, and has been generally reduced along the driveway because of the redevelopment. Peak VxD at the Site entrance still exceeds 1.0, however, along the southern parking space area, VxD is typically reduced from 0.85 m^2/s in pre-developed conditions to less than 0.27 m^2/s.

Downstream, VxD of the secondary overland flow path along the southern boundary has reduced, however, instead VxD has been increased downstream of the proposed outlet and dissipation structure.

6.2.4 Post-Developed Peak Flood Depths PMF

Flood affectation in terms of depths, extents and level contours is shown for the PMF design event on Figure_A 11. Peak flood depths have reduced slightly in Beach Street due to the slightly wider entrance being proposed.

Upon entering the Site, after water is captured in the proposed grated inlet structure, flood depths reach up to 250 mm along the southern car parking spaces. Flood water is expected to no longer completely inundate the northern car parking area, or inundate the floor level of 'The Stables' building under modelled conditions. It still overtops the kerb towards Cliffbrook House, though only very shallow flow of less than 50 mm is expected to pass around the southern side of Cliffbrook House.

The highest flood level along the entrance to the basement car park is 30.180 m AHD. It is reasonable to expect flood water to enter the basement car park during the PMF event.

Sheeting flood water is expected to pass around the north side of the basement car park entrance then continue eastward towards the pedestrian staircase. Water is not expected to enter the ground floor level on the western side of the proposed building at this location. At the base of the staircase flood water is expected to reach a depth of up to 161 mm and a flood surface level of 27.036 m AHD, which will be higher than the lower ground floor around this location (27.00 m AHD).

Adjacent to the eastern side of the proposed building, flood water is ranges from 4 mm up to 161 mm in depth, with a flat flood surface level of 27.036 m AHD, which will be higher than the lower ground floor around this location (27.00 m AHD).

6.2.5 Post-Developed Peak Flood Velocities PMF

Peak velocities for the PMF event are shown in Figure_A 14. Through the entrance, peak velocities no longer exceed 10.0 m/s, however, they are still considerably high. Peak velocities typically reach up to 6.0 m/s along the southern kerb until reducing to 4.6 m/s by the weir.

Within the primary overland flow path peak velocities typically range from 3.0 to 4.0 m/s, however, velocities reach 5.0 m/s near the proposed outlet and dissipation zone.

Peak velocities in the secondary overland flow path typically range from 1.5 to 3.5 m/s in the grassed area, and are typically less than 1.3 m/s along the southern boundary edge in the vegetated area.

6.2.6 Post-Developed Peak Flood Velocity Depth Product PMF

Peak VxD for the post-developed PMF event is shown in Figure_A 17, and has been generally reduced along the driveway because of the redevelopment. A peak VxD of 2.7 m^2/s is reached at the Site entrance, and VxD typically exceeds 1.0 along the remainder of the driveway and southern parking spaces, with peak values of VxD reaching 1.9 m^2/s. Both the primary and secondary overland flow paths typically exceed 1.0 m^2/s, with peak VxD values of 1.6 m^2/s and 2.0 m^2/s respectively.

6.3 Water Entry into Buildings and Basement Car Park

Authorities typically require a freeboard between proposed habitable floor levels and flood levels. For 1% AEP storm events, freeboard is usually 0.5 m for habitable areas subject to significant flooding risk and 0.15 m for basement car park entrances. Authorities sometimes ignore areas where flood depth is 50 mm or less, and consider it as sheet flow or a low flood risk area.

Contained in the table below are the existing building and proposed building floor levels, and modelled 1% AEP and PMF event flood levels where applicable.

Freeboards (floor level – peak flood level) for the buildings on the proposed ground floor and the basement car park entrance are above these limits.

Table 2 Building Entry Levels and Flood Levels

Location	**Floor Level m AHD**	**Peak Flood Level m AHD**	
		1% AEP Event	**PMF Event**
Cliffbrook House (Existing Building) Western Entrance	30.975 / 31.155[1]	- [2]	31.05
Cliffbrook House (Existing Building) Southern Entrance	31.190	- [2]	30.85
Cliffbrook House (Existing Building) Northern & Eastern Entrance	30.790	- [2]	- [2]
The Stables (Existing Building)	31.300	- [3]	31.340
Proposed Building Ground Floor Western Entrance	30.600	- [4]	30.285
Proposed Building Lower Ground Floor Eastern Entrance	27.000	26.925[5]	27.036
Basement Car Park Entrance	30.150	30.000[6]	30.200[6]

[1] The first room is lower with a floor level of 30.975, although connected rooms step up to 31.155.

[2] There is no modelled flood water near the entrances to this building.

[3] There is no modelled flood water near the entrances to this building. There is still likely to be some overland flow from direct rainfall within the Site that could penetrate the entrances to this building.

[4] There is no modelled flood water near the entrance to this building.

[5] Modelled flood water is expected to back up from downstream up to a level 75 mm lower than the proposed lower ground floor level, however, flood depth is typically 2 mm to 49 mm deep at these locations. A depth of less than 50 mm is generally considered as general sheet flow, rather than flood flow.

[6] The modelled levels are assuming that the weir from the driveway is unblocked (no vehicles washed against the bollards) leading into the primary overland flow path. Although it is unlikely for a vehicle to be washed against the bollards in the 1% AEP event, it is reasonably possible that it does happen during a rare PMF event. If this occurs, it would be reasonable to expect that flood levels would be higher, leading to flood water entering the basement car park.

Even with these levels typically set above the 1% AEP flood, the buildings and basement car park can still be flooded in rare events up to the PMF. Given the number of occupants in the building, a Flood Evacuation Plan must be prepared as part of the development's Emergency Management Plan, and must address the possibility of the basement filling with water in a rare storm event.

6.4 Climate Change

The Floodplain Development Manual 2005 (FDM) and ARR requires that Flood Studies and Floodplain Risk Management Studies consider the impacts of climate change on flood behaviour. In Section 4.1, it was established that an 18.6% increase in rainfall depth is predicted to occur by the year 2090. This was modelling in DRAINS using a 1.186 multiplier applied to the BOM IFDs.

In addition to increasing rainfall depths due to climate change, the effects of sea level rise also need to be considered, which generally includes allowing for up to a 1 m rise by 2100. As the Site is located significantly above 1 m AHD, and the overland flows will not be impacted by downstream tides or tailwater conditions, assessment of sea level rise is not required for this study.

Table 3 compares the 1% AEP flood levels to the estimated levels due to climate change. An increase in rainfall of 18.6% would increase the 1% AEP flood levels by 15 mm against the lower ground floor entry points, and by 80 mm at the entrance to the basement car park. These increased flood levels are still below the proposed levels of the lower ground floor and entry to the basement car park, however, freeboard would be reduced.

Table 3 Increase to 1% AEP Flood Levels due to Climate Change

Location	**Floor Level m AHD**	**Peak Flood Level m AHD**	
		1% AEP Event	**1% AEP Event + CC**
Cliffbrook House (Existing Building) Western Entrance	30.975 / 31.155[1]	-	-
Cliffbrook House (Existing Building) Southern Entrance	31.190	-	-
Cliffbrook House (Existing Building) Northern & Eastern Entrance	30.790	-	-
The Stables (Existing Building)	31.300	-	-
Proposed Building Ground Floor Western Entrance	30.600	-	-
Proposed Building Lower Ground Floor Eastern Entrance	27.000	26.925	26.937
Basement Car Park Entrance	30.150	30.000	30.080

6.4.1 Peak Flood Hazard Classification 1% AEP + Climate Change

Figure_A 19 provides the hazard classification for the post-developed 1% AEP + climate change event based on Figure 48. Compared to Figure_A 18, hazard classifications do not significantly change, however, when comparing Figure_A 15 and Figure_A 16, velocities along the southern parking area have increased from almost 3 m/s up to 3.8 m/s, which would increase the risk to pedestrians and vehicles above ARR 2016 guideline limits.

7 FLOOD HAZARD ASSESSMENT

7.1.1 Defining Peak Flood Hazard Classification

The hydraulic categories, namely floodway, flood storage and flood fringe, are described in the Floodplain Development Manual 2005 (FDM). However, there is no technical definition of hydraulic categorisation that would be suitable for all catchments, and different approaches are used by different consultants and authorities, based on the specific features of the study catchment.

As with hydraulic categories, hazard classification plays an important role in informing floodplain risk management in an area. Previously, hazard classifications were binary – either Low or High Hazard as described in the FDM. However, in recent years there has been a number of developments in the classification of hazard. The publication '*Updating National Guidance on Best Practice Flood Risk Management'* (D. McLuckie et. al., 2014) provides revised draft hazard classifications which add clarity to the hazard categories and what they mean in practice.

These draft hydraulic categories have been defined by the following criteria, which corresponds with the FDM, however, hazard categories have been grouped based on clearly defined consequences. The classification is divided into six categories (Table 4 and Figure 48) which indicates the restrictions on people, buildings and vehicles.

A constraint of these draft hazard classifications is that velocities must not exceed 2 m/s under any condition without automatically raising any hazard classification to H5 'Unsafe for all pedestrians and vehicles'. The 2 m/s velocity limit was initially proposed in 1986 by the Department of Public Works (DPW), retained in the FDM 2005, and adopted again in these draft hydraulic categories (D. McLuckie et. al., 2014).

As the classifications are aimed at providing baseline information for general consideration as part of an initial scoping exercise for a floodplain management study, this limit is understandable. With the focus of the current study relating to specific detailed aspects within a Site, such as modelling the flows within the kerb and gutter along the southern parking spaces where the driveway commences at 10% grade then flattens out to 5%, it is difficult to achieve a velocity of less than 2 m/s for flow depths greater than 40 mm. A velocity limit of 2 m/s would require the majority of the overland flows to be piped.

Another current guideline (ARR 2016, Book 6, Chapter 7) combines over four decades of Australian and international studies and laboratory experiments previously documented by ARR in the '*Revision Project 10: Appropriate Safety Criteria for People (Cox et al., 2010)*' and '*Revision Project 10: Appropriate Safety Criteria for Vehicles - Literature Review (Shand et al. 2011)*'. It provides recommendations of velocity depth product ratios, maximum depths, and maximum velocities for various categories of people and vehicles (Table 5, Figure 49, Table 6, and Figure 50). The prescribed velocity depth product ratios and maximum depth limits are similar to the classifications by D. McLuckie et. al., 2014, however, maximum velocity limits of 3 m/s are recommended.