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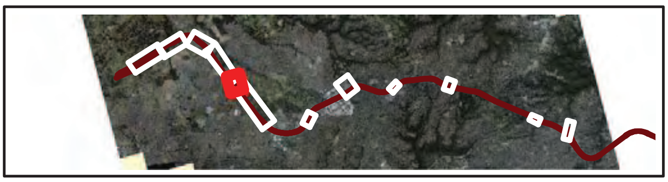

North West Rail Link EIS
Change in Peak Flow Velocities - Proposed Design (100 Year ARI)
Elizabeth Macarthur Creek
Source: AECOM

0 0.03 0.06 0.12 km

25 SEP 2012
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Fig. **19**

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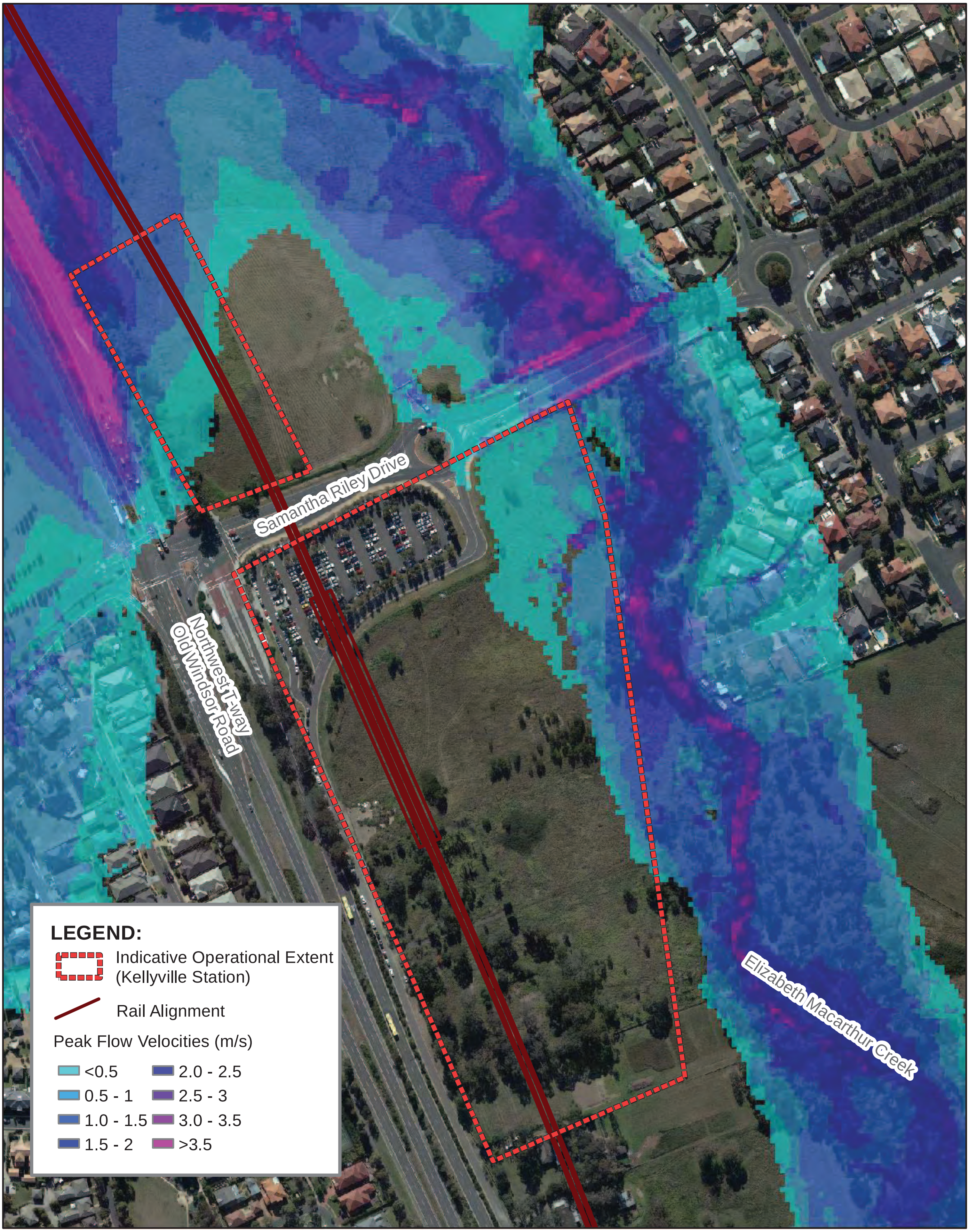

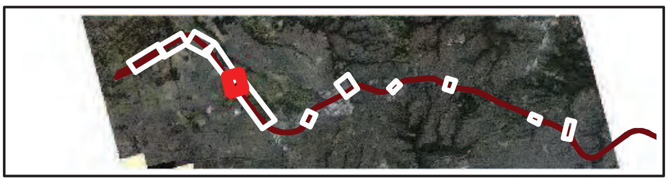

North West Rail Link EIS
Peak Flow Velocities - Existing (PMF)
Elizabeth Macarthur Creek
Source: AECOM

0 0.03 0.06 0.12 km

25 SEP 2012
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Fig. **20**

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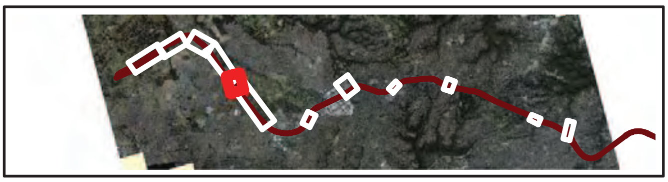

North West Rail Link EIS

Change in Peak Flow Velocities - Proposed Design (PMF)

Elizabeth Macarthur Creek

Source: AECOM

0 0.03 0.06 0.12 km

25 SEP 2012

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Fig. **21**

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5.1.3.5 Caddies Creek Confluence with Tributary 5 and Elizabeth Macarthur Creek (Kellyville Station to Windsor Road)

Between Kellyville Station and Windsor Road the rail alignment traverses the broad floodplain of Caddies Creek including its confluence with Elizabeth Macarthur Creek and Caddies Creek Tributary 5. The creek lines in this area are moderately incised with well vegetated main channel and overbank areas. Windsor Road lies to the west of the rail alignment while to the east of the alignment and creek is residential development.

The rail alignment in this area consists of a viaduct elevated above the floodplain. Based on the concept design it is anticipated that the viaduct would consist of box section spans, typically 36m in length, supported by columns and headstocks. Longer spans would be required to traverse key infrastructure such as road crossings.

Kellyville Station Precinct includes an area north of Samantha Riley Drive that is outside the 100 year ARI flood extent but would be inundated in the PMF event.

The impacts of the viaduct structure were assessed as part of EIS 1. For the purposes of EIS 2 the viaduct structure has been modelled in combination with station precinct works to assess the combined impacts of EIS 1 and EIS 2 works.

Detailed hydraulic modelling has been carried out to assess the potential impacts of the proposed viaduct arrangement and station precinct works on the existing flood regime. The hydraulic modelling undertaken is discussed in Appendix B. The results of this assessment are summarised below.

Potential flood impacts of the viaduct arrangement have been assessed based on the current concept design arrangement, consisting of twin 1.8m diameter concrete columns at each span support. Due to limited details currently available on finished levels within the precinct the extent of precinct works within the floodplain have been assumed to be located above the existing flood levels. This would provide an upper bound estimate of potential impacts associated with the precinct works.

Flood Level Impacts

Modelled flood impacts are shown on Figure 22 for the 100 year ARI event. The results show that there would be localised flood level impacts of up to 0.06m in the 100 year ARI event around the viaduct columns. Within the adjacent residential development the flood level impacts would be up to 0.04m but typically less than 0.02m. There is the potential for localised impacts of 0.05m in areas that are currently sensitive to flooding. These impacts could be offset by local flood mitigation works such as bunding or levees. The design of these overbank works depends on the final location, size, shape and spacing of the viaduct piers to be determined during the future detailed design stages.

Flood impacts were also assessed for the Probable Maximum Flood (PMF) to identify the implications for regional flooding during events in excess of the 100 year ARI event (refer Figure 23). The results indicate that flood level impacts would generally be less than 0.1m on the floodplain around the confluence of Caddies and Elizabeth Macarthur Creeks.

However, the precinct layout assumed for the modelling would result in flood level impacts of up to 0.5m at Old Windsor Road and the Transitway near the intersection with Samantha Riley Drive. While Old Windsor Road and the Transitway are already flooded in the PMF under existing conditions, there is the potential for parts of Samantha Riley Drive to flood that are flood free under existing conditions. Apart from this, the overall flood extent would not increase significantly in a PMF event and so increases in flooding due to the proposed works would have negligible impact.

Flow Velocity Impacts

Peak flow velocities in the 100 year ARI event under existing conditions are generally low in overbank areas of the floodplain (less than 1m/s). Within the main channel, peak flow velocities are in the order of 1-2m/s, but can be up to 3.5m/s in isolated locations, depending on local channel controls (refer Figure 24).

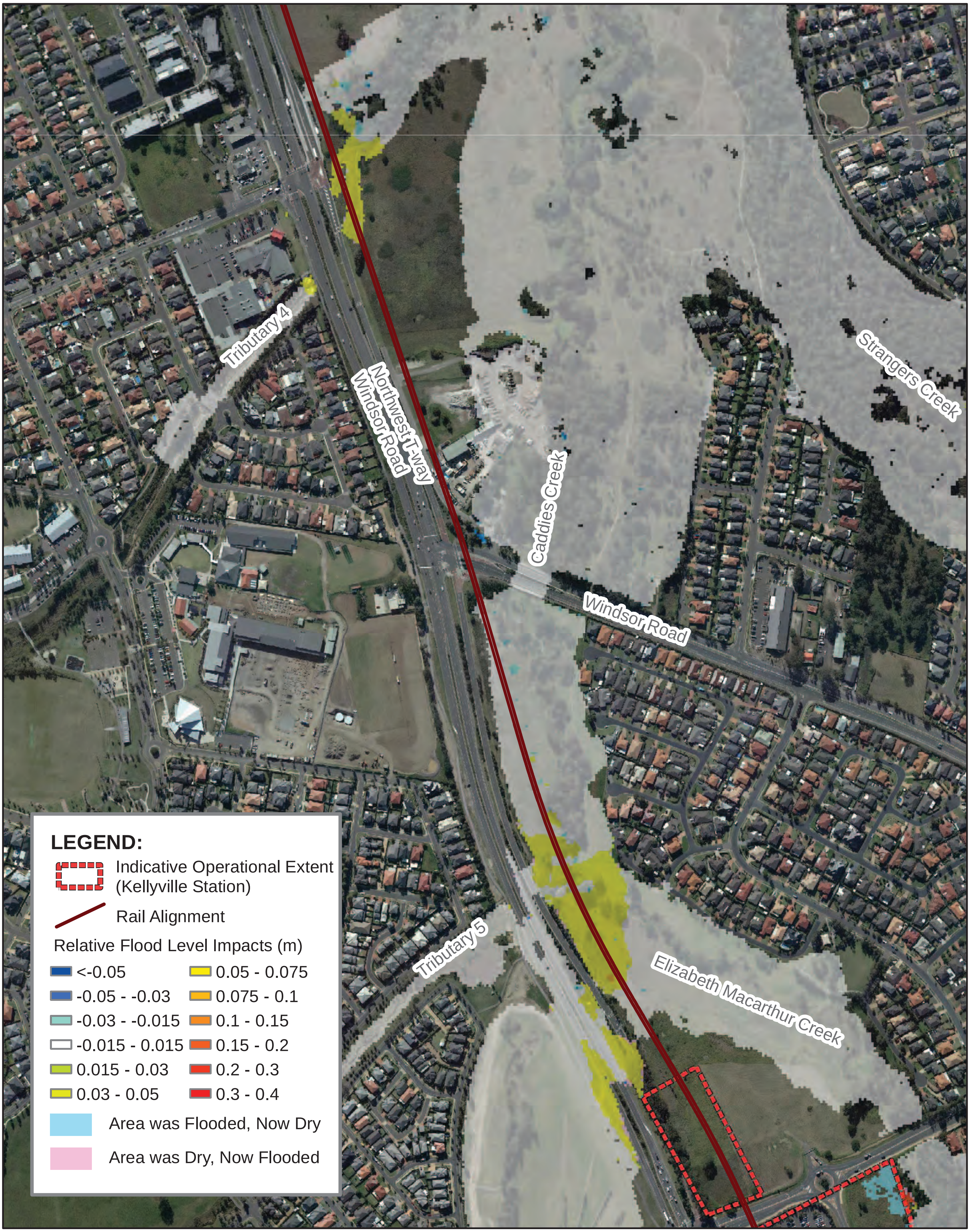
The flood modelling shows that there would be localised increases in peak flow velocity in the 100 year ARI event, particularly on the floodplain around the confluence of Caddies Creek, Elizabeth Macarthur Creek and Caddies Creek Tributary 5 (refer Figure 25 for peak flow velocity impacts). Increases in velocity would be typically less than 0.1m/s but up to 0.3m/s. However, peak velocities in these areas would still be relatively low (less than 1.5m/s in the 100 year ARI event) and hence unlikely to cause excessive erosion.

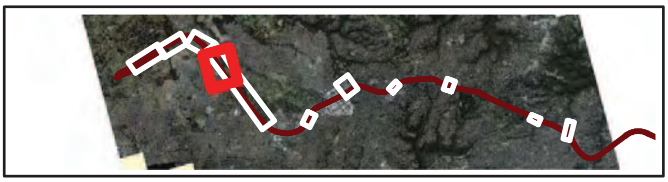
Velocities would also likely increase locally around the viaduct piers. This would require appropriate scour protection, especially around piers located close to the main channels where peak flow velocities are likely to be high (greater than 2m/s).

Peak flow velocities in the PMF are in the order of 1-2m/s in overbank areas, but can be greater than 4m/s where roads are overtopped (refer Figure 26). Peak flow velocities are likely to increase by up to 0.5m/s along Old Windsor Road and the Transitway (refer Figure 27). However, peak flow velocities would already be high under existing conditions in most areas affected and so the relative change in velocity would not be significant. The greatest impacts would be expected downstream of the Transitway near the proposed Kellyville Station precinct. Peak flow velocities could potentially increase from 2.5 to over 3.5m/s. The concept design includes only a parking area for the Kellyville Station precinct north of Samantha Riley Drive.

Conclusion

Overall, the modelled impacts show that it would not be feasible to raise the precinct works completely above existing flood level without having adverse impacts on the surrounding environment. In particular, it would be necessary to manage impacts on Old Windsor Road and the Transitway for flooding in excess of the 100 year ARI event. For the area of the precinct north of Samantha Riley Drive provision for overland flows in the PMF would need to be made to reduce the potential impacts.





North West Rail Link EIS
Relative Flood Level Impacts (100 Year ARI)
Caddies Creek
Source: AECOM

0 0.05 0.1 0.2 km

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Fig. **22**

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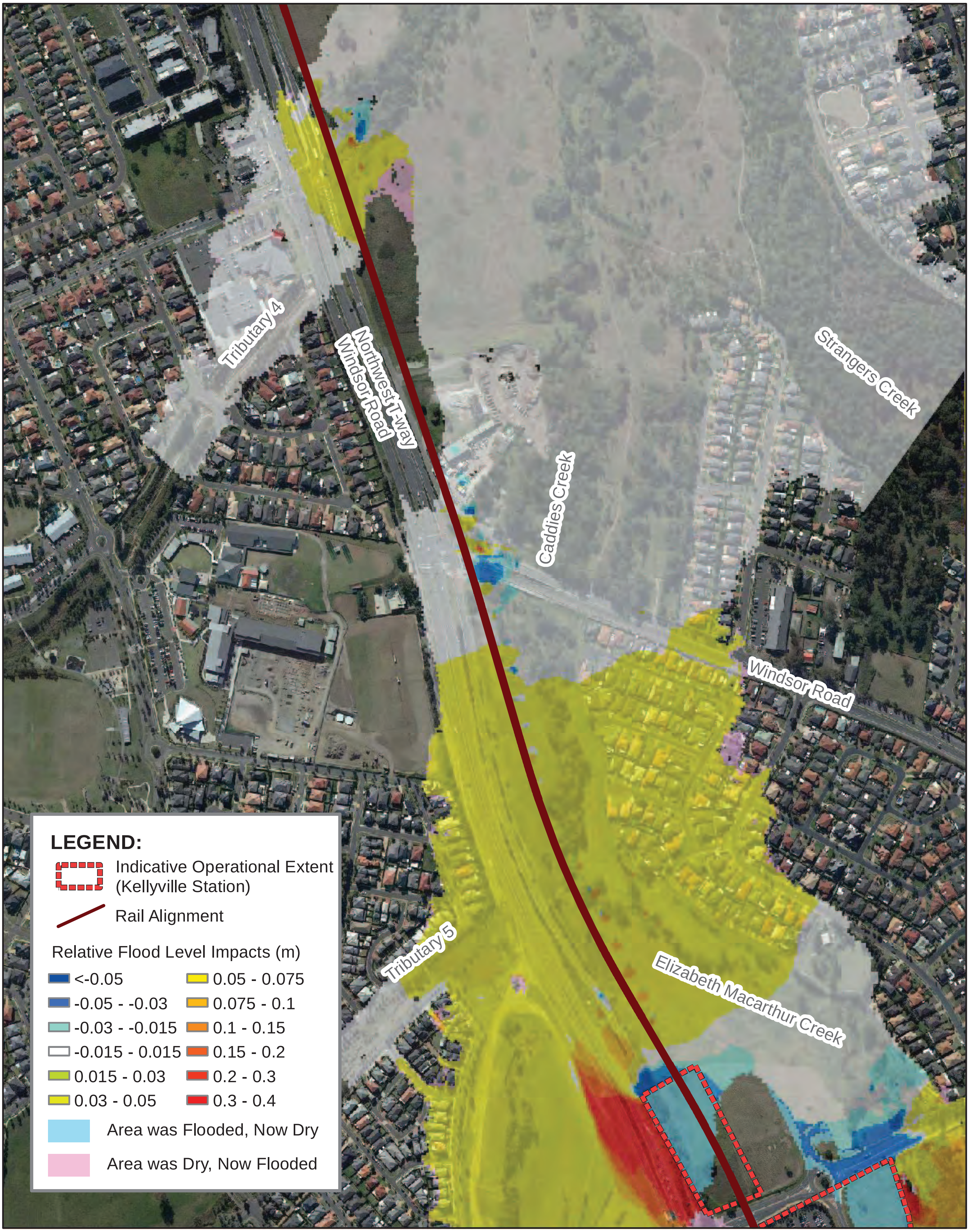

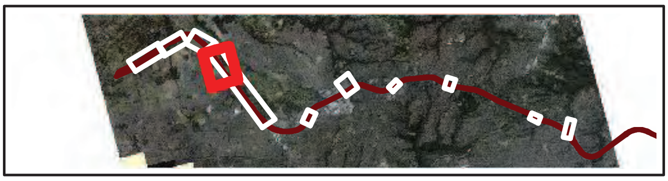

North West Rail Link EIS
Relative Flood Level Impacts (PMF)
Caddies Creek
Source: AECOM

0 0.05 0.1 0.2 km

25 SEP 2012
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Fig. **23**

AECOM P:\60220686_NWRL\4. Tech work area\4.3. Engineering\Flooding and Drainage\GIS\Maps\EIS\Figure23_EA_V2.mxd Drafted 25/09/2012

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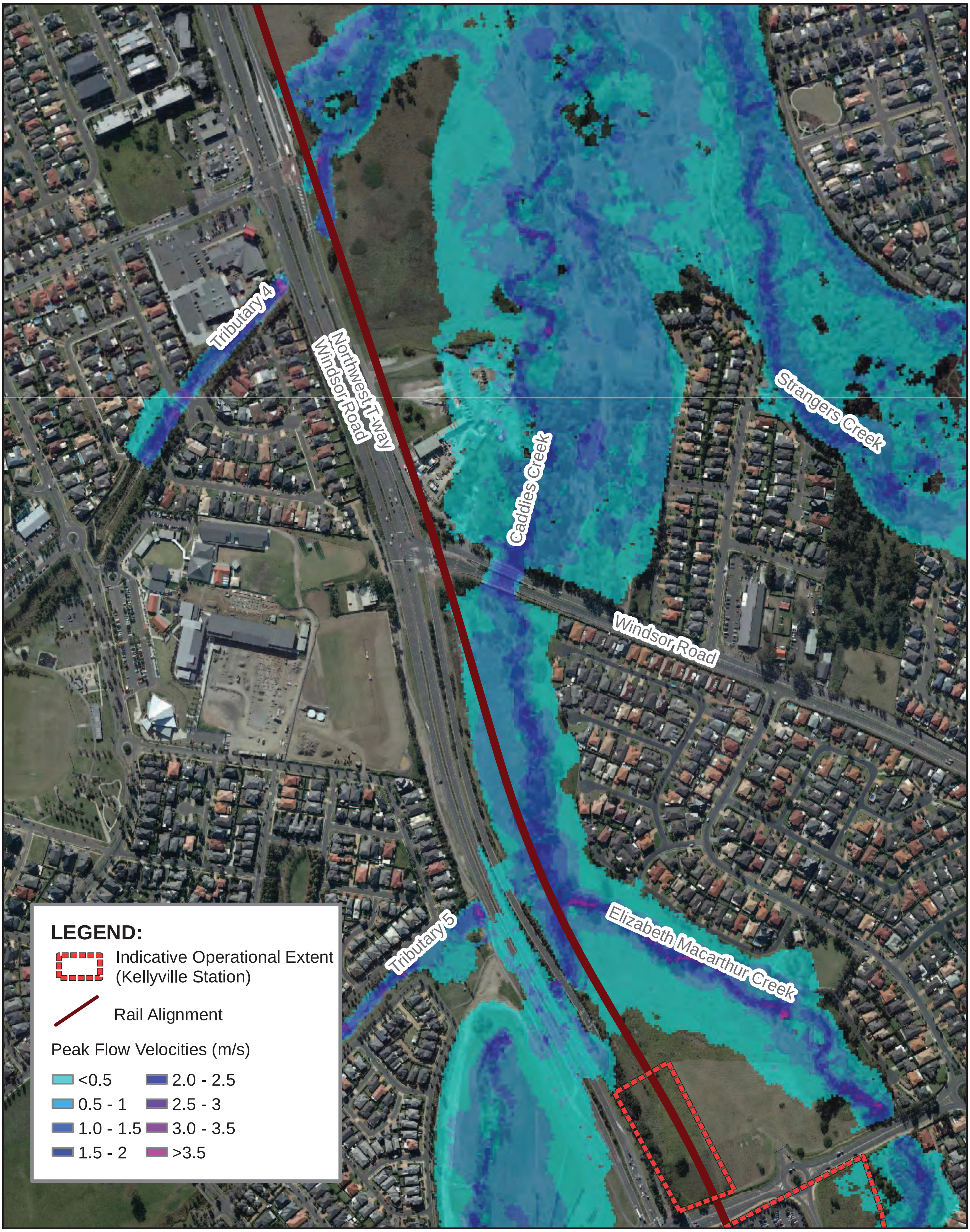

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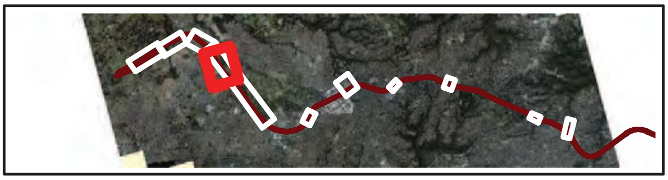

North West Rail Link EIS
Peak Flow Velocities - Existing (100 Year ARI)
Caddies Creek
Source: AECOM

0 0.05 0.1 0.2 km

25 SEP 2012

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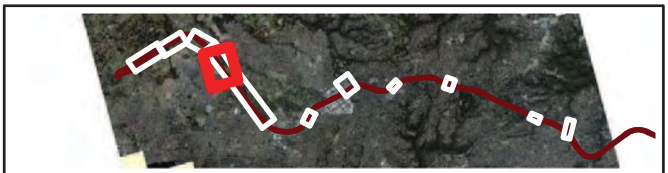
Fig. **24**



North West Rail Link EIS
Change in Peak Flow Velocities - Proposed Design (100 Year ARI)
Caddies Creek
Source: AECOM

0 0.05 0.1 0.2 km

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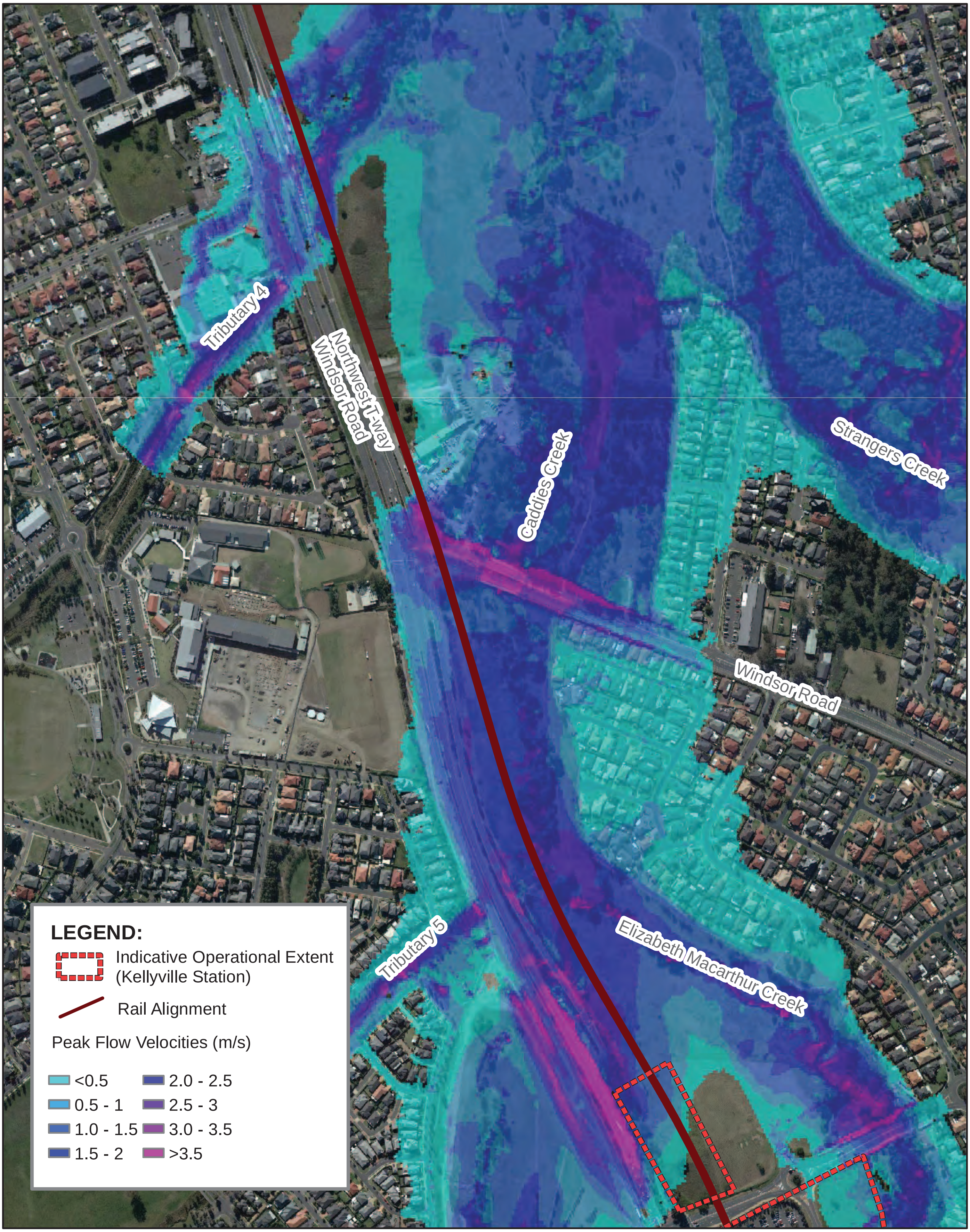
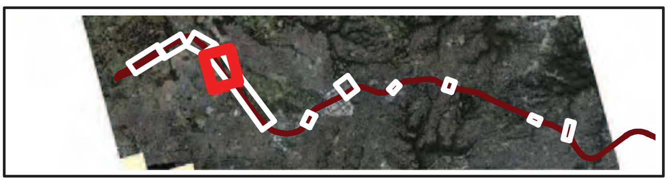
Fig. **25**

AECOM





North West Rail Link EIS
Peak Flow Velocities - Existing (PMF)
Caddies Creek
Source: AECOM

0 0.05 0.1 0.2 km

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Fig. **26**

AECOM P:\60220686_NWRL\4. Tech work area\4.3. Engineering\Flooding and Drainage\GIS\Maps\EIS\Figure26_EA_V2.mxd Drafted 25/09/2012

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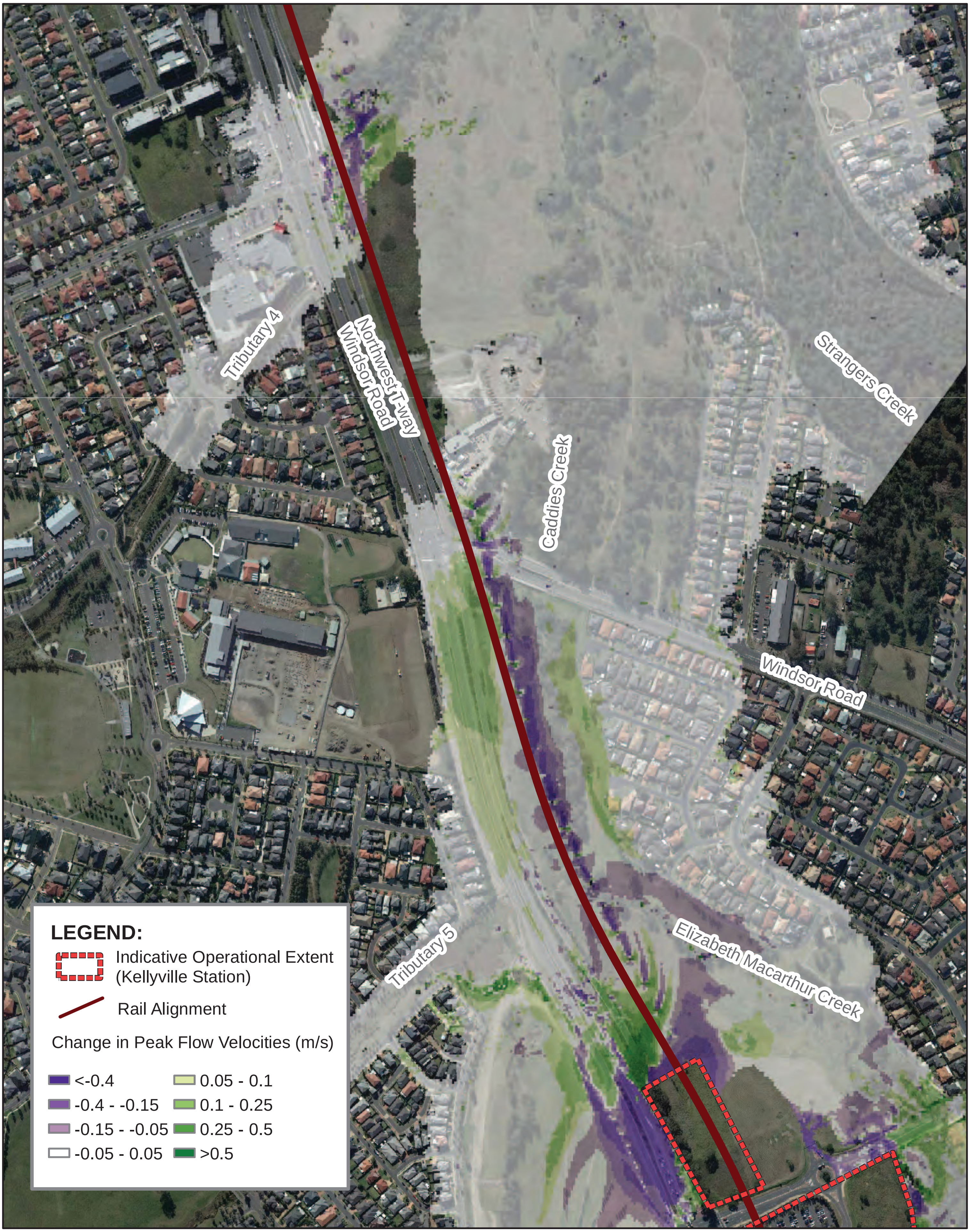

North West Rail Link EIS
Change in Peak Flow Velocities - Proposed Design (PMF)
Caddies Creek
Source: AECOM

0 0.05 0.1 0.2 km

25 SEP 2012
60224114
Fig. **27**

AECOM P:\60220686_NWRL\4. Tech work area\4.3. Engineering\Flooding and Drainage\GIS\Maps\EIS\Figure27_EA_V2.mxd Drafted 25/09/2012

5.1.3.6 Caddies Creek Tributary 4

There would be localised impacts at Tributary 4 in the 100 year ARI event as a result of viaduct piers located in the floodplain. However, the cross-sectional area of structure columns within the floodplain relative to the total floodplain area is relatively small and so, based on hydraulic modelling undertaken for Caddies Creek, the potential impacts are considered to be negligible for both the 100 year ARI and PMF events (refer Figure 22 and Figure 23).

Given the size of the waterway relative to the area of piers within the floodplain, changes in velocities resulting from the proposed bridge crossing are estimated to be generally negligible. There would however be localised increases in velocity around piers that would require appropriate scour protection measures in accordance with normal bridge design practices.

No station precinct works are proposed at Caddies Creek Tributary 4 and therefore there would be no additional impacts as a result of the EIS 2 works.

5.1.3.7 Caddies Creek Tributary 3

At Tributary 3, flood studies show that Windsor Road is not overtopped in a 100 year ARI event. Consequently flows discharging from the Windsor Road culverts in the 100 year ARI event are confined to the channel downstream, resulting in a relatively confined flow width. Based on the 100 year ARI flood width the viaduct piers are expected to span the 100 year ARI flood extent. Consequently, no impacts are expected for flooding up to the 100 year ARI event.

The precinct would be susceptible to inundation from flows that overtop Windsor Road at the Tributary 3 culverts. Overtopping of Windsor Road would occur in the PMF event and the southern end of the precinct would be inundated. The area affected currently comprises carpark, T-way interchange and local access roads for the Rouse Hill town centre. Finished levels are expected to be largely unchanged under the proposed precinct design. Consequently, flood impacts in the PMF are not expected to be significant.

5.1.3.8 Second Ponds Creek

Second Ponds Creek at the NWRL alignment has an upstream catchment area in the order of 620 hectares. The catchment has undergone significant urban development over recent years and is ongoing. At present parts of the catchment in the immediate vicinity of the Project corridor are still largely undeveloped and consist mainly of rural residential. Existing areas of rural development are earmarked for urbanisation as part of the Area 20 Precinct.

Potential flood impacts associated with the proposed bridge/viaduct structure have been assessed under EIS 1 based on the concept design arrangement.

No station precinct works are proposed within the Second Ponds Creek floodplain and therefore there would be no additional impacts as a result of the EIS 2 works.

5.1.3.9 First Ponds Creek

The proposed location of the stabling facility at Tallawong Road is currently under design development. The latest concept design, which shows the stabling facility being located between Tallawong Road and First Ponds Creek, has been adopted for the purposes of identifying potential flood risks and impacts. Under this arrangement the facility would be located outside the PMF flood extent of First Ponds Creek. Therefore based on the current concept design there would be no flood impacts or risks expected at this precinct apart from potential local drainage and overland flow issues.

5.1.4 Floodplain Storage

5.1.4.1 Operational Phase

Impacts of major civil works (rail embankments, viaduct and bridges) on floodplain storage were assessed under EIS 1. As has been identified, the EIS 2 works would involve some station precinct works within the floodplain. In these areas it will be necessary to provide a balance of cut and fill up to the 100 year ARI flood level to minimise impacts on floodplain storage.

5.1.4.2 Construction Phase

Impacts on floodplain storage associated with the construction of major civil works, such as the fill embankment, viaduct and bridge structures were addressed in EIS 1. This assessment included a preliminary assessment of the potential impacts of temporary haul roads and working pads associated with the construction of the bridges and viaduct.

Temporary filling within the floodplain for the construction of the stations, precincts and ancillary facilities proposed under EIS 2 will be minimal. Filling within the floodplain will be removed at the completion of construction to ensure that there are no long term impacts.

5.1.5 Stormwater Quantity

5.1.5.1 Operational Phase

The proposed stations and rail infrastructure will alter the percentage impervious area within the catchments that the project traverses. This will lead to increased volumes of runoff and changes in catchment response times. Appropriate measures such as on site detention (OSD) facilities and/or water sensitive urban design (WSUD) features will be provided to mitigate potential flood impacts.

Between Epping and Showground OSD will be required in accordance with Hornsby Shire Council and The Hills Shire Council. A summary of requirements is provided in Table 9.

Table 9 OSD Requirements – Epping to Showground

Site	Existing Site	Proposed Site	OSD requirements
Epping Services Facility	Commercial development.	Facility building with vehicular access and carparking.	No significant increase in imperviousness or change in development type. Therefore, need for OSD is subject to confirmation with Council. If required, OSD would need to be provided in accordance with Hornsby Shire Council OSD policy.
Cheltenham Services Facility	Park with oval, netball court and carparking.	Facility to be integrated with existing land use. Additional facility building, carparking and road modifications.	No significant increase in imperviousness or change in development type. Therefore, need for OSD is subject to confirmation with Council. If required, OSD would need to be provided in accordance with Hornsby Shire Council OSD policy.
Cherrybrook Station	Largely undeveloped or low density residential.	Station precinct with carparking, transport interchange, commercial buildings, local roads and pedestrian plazas surrounding station.	Significant increase in impervious area and change in land use. OSD will be required in accordance with Hornsby Shire Council OSD policy.
Castle Hill Station	Arthur Whitling Park	Station and facilities building to integrated with existing park. Modifications to existing roads.	Minor increase in area of impervious and no substantial change to existing function of the site. Therefore, need for OSD is subject to confirmation with Council. If required, OSD would need to be provided in accordance with The Hills Shire Council OSD policy.
Showground Station	Part of Castle Hill Showground and Council Depot.	Station precinct with carparking, transport interchange, commercial buildings, local roads and pedestrian plazas surrounding station.	Significant increase in impervious area and change in land use. OSD will be required in accordance with The Hills Shire Council OSD policy.

Between Norwest Station and Cudgegong Road Station, the station precincts fall within an area where a regional stormwater management strategy has been implemented as part of the North West Growth Centre. This regional strategy is outlined in the report titled *Rouse Hill Stage 1b Area Trunk Drainage Strategy* (GHD, 1998) and was further reviewed and updated in *Rouse Hill Integrated Stormwater Strategy Review – Hydrology, Hydraulics and Water Quality Review* Sinclair Knight Merz (2009).

A key objective of the regional trunk drainage strategy for the North West Growth Centre is that 100 year ARI peak flows discharging to areas further downstream under ultimate conditions do not exceed existing peak flows. To achieve this, the strategy is based on a regional detention basin approach catering for all development within the catchment, rather than separate sub-division detention basins or individual OSD measures. However, if runoff from a particular site increases peak flows above existing such that it would impact on existing development then OSD would need to be considered.

- **Norwest Station**: The existing site has approximately 75% impervious surfaces. The proposed precinct layout caters for the same hardstand area. OSD is not required as there will be no change in peak flow runoff from site.
- **Bella Vista Station**: Impervious cover for the sub-catchment that the precinct is located in is estimated to increase from 5 to 12% with the station precinct, but peak flows in Elizabeth Macarthur Creek are not increased over existing peak flows and OSD is therefore not required.
- **Kellyville Station**: Impervious cover for the sub-catchment that the precinct is located in is estimated to increase from 35 to 62% within the station precinct. However, increases in paved area are consistent with the regional detention strategy and peak flows in Elizabeth Macarthur Creek are not increased over existing conditions. Consequently, OSD is not required.
- **Rouse Hill Station:** The existing site has approximately 90% impervious surfaces. The proposed precinct layout caters for the same hardstand area. The proposed precinct is therefore consistent with the regional detention strategy and would have no impacts on existing flows. Therefore OSD is not required.
- **Cudgegong Station**: Impervious cover for the sub-catchment that the precinct is located in is estimated to increase from 17 to 40%. However, increases in paved area are consistent with the regional detention strategy and peak flows in Second Ponds Creek are not increased over existing conditions. Consequently, OSD is not required.
- **Tallawong Stabling Facility**: The stabling facility falls within the Riverstone East Precinct in the North West Growth Centres. Masterplanning for this precinct is still underway and while it is anticipated that this will make provision for the NWRL, this has not yet been confirmed. The existing site is currently rural residential with a low area of imperviousness (approximately 5%). The proposed stabling facility layout will result in an increase in impervious area to approximately 25%. Hydrologic modelling of the proposed stabling facility layout has identified that the increase in impervious area would result in negligible change (less than 2%) in peak flows in First Ponds Creek.

While the regional and local detention strategies identified above would manage flooding in larger events, up to the 100 year ARI event, there is a risk of increased volume and velocity of flows to receiving waterways during more frequent rainfall events. This could lead to an exacerbation of erosion and the mobilisation of sediments. This will be managed by implementing appropriate water sensitive urban design (WSUD) measures such as grassed swales, bioretention systems and use of rainwater harvesting at buildings. Where discharge is to an existing stormwater network, the capacity of the existing network will need to be assessed to ensure that the system can cope with additional flows.

5.1.5.2 Construction Phase

Proposed works during construction, particularly earthworks and temporary access roads, will alter the extent of impervious area and catchment response times. Potential impacts would be offset by the provision of erosion and sediment control measures, such as sediment basins and bunded swales, which are designed to control the discharge of runoff from the site.

5.1.6 Construction Impacts

The construction of the stations, precincts and services facilities will follow the construction of major civil works covered under EIS 1. The location of the construction sites for the EIS 2 works would be consistent with the sites required for the major civil construction works addressed in EIS 1 and listed in Table 10.

Table 10 Major Waterway Catchments and Construction Sites

Catchment	Construction Site
Devlins Creek	1. Epping Services Facility
	3. Cheltenham Services Facility
Pyes Creek	4. Cherrybrook Station
Cattai Creek	5. Castle Hill Station
	6. Showground Station
Strangers Creek	7. Norwest Station
Elizabeth Macarthur Creek	8. Bella Vista Station
	9. Balmoral Road
	10. Memorial Avenue
	11. Kellyville Station
Caddies Creek (including Tributaries 3, 4 and 5)	12. Windsor Road/Old Windsor Road
	13. Old Windsor Road/Whitehart Drive
	14. Rouse Hill Station
	15. Windsor Road Viaduct
Second Ponds Creek	16.Windsor Road Viaduct to Cudgegong Road
First Ponds Creek	17. Cudgegong Road and Tallawong Stabling Facility

Note: Site 2 described within EIS 1 was deleted as part of the EIS 1 preferred infrastructure report

A detailed description of the construction activities that would be undertaken as part of EIS 2 works is provided in Chapter 7 of the main EIS 2 report. Activities would include station construction and fit-out, station precinct works, services and stabling facility construction and fit-out, tunnel, at-grade and viaduct systems fit-out and testing and commissioning. Of these activities it is the works within the station precincts, services facilities and stabling facility that have the potential for flood related impacts. This relates to works within Sites 1 to 8, 11, 14 and 17. Potential flood impacts within these sites are discussed in the following subsections.

5.1.6.1 Below Ground Stations and Facilities - Epping Station to Bella Vista Station

For the tunnel section of the NWRL between Epping and Bella Vista Station, flood inundation of excavations for below ground stations and services facilities could lead to flooding of the tunnels and result in damage to works, delays in construction program and risk to personal safety. Refer to Table 11 for a summary of below ground stations and services facilities.

Table 11 Summary of Flooding Potential at Proposed Under Ground Stations and Associated Precincts during Construction

Construction Site	Flooding Potential
1. Epping Services Facility	Small portion of site adjacent to Beecroft Road tributary is flood affected in the 20 year ARI event. Refer to Figure 3 for flood extent mapping.
3. Cheltenham Services Facility	M2 Motorway is located between the site and Devlins Creek. Consequently, site is not affected by mainstream flooding up to the 100 year ARI. Local drainage flows across site from northeast.
4. Cherrybrook Station	Site is not affected by mainstream flooding. However, local overland flowpath runs south to north across the site and proposed station portal.
5. Castle Hill Station	Site is located at top of catchment and is not affected by mainstream flooding.
6. Showground Station	Part of site spans Cattai Creek and is flood affected. Refer to Figure 6 for flood extent mapping.
7. Norwest Station	Site is not affected by mainstream flooding. However, local drainage runs west to east along Norwest Boulevard.
8. Bella Vista Station	Small portion of site adjacent to Elizabeth Macarthur Creek is flood affected in the 20 year ARI event. Refer to Figure 9 for flood extent mapping.

During the construction of the below ground stations and facilities and associated precincts, it will be necessary to ensure that the potential for ingress of floodwaters into the sites is appropriately managed especially at the entries to the underground sites.

At all sites there would be potential for local runoff to enter the stations/facilities and this would need to be addressed through local stormwater management of the site.

The flood assessment has identified that the construction sites at Epping Services Facility, Showground and Bella Vista have the greatest potential risk of flood affectation. The layout of the sites will need to be developed taking into consideration the nature and potential risk of flooding.

The flood standard adopted at each station/facility during construction will need to be developed taking into consideration the duration of construction, the magnitude of inflows and the potential risks to the project works, facilities and personal safety.

5.1.6.2 Above Ground Stations and Facilities

Above ground stations and precincts are proposed for Kellyville (Site 11), Rouse Hill (Site 14) and Cudgegong Road (Site 16), as well as the Tallawong Stabling Facility (Site 17). The above ground stations and stabling facility are generally located outside the 100 year ARI flood extent and therefore flooding is not expected to pose a significant risk during construction of these precincts.

5.1.7 Potential Impacts Due to Climate Change

Scientific research into the potential impacts of climate change has been rapidly evolving over recent years. Latest research indicates that climate change is likely to result in more frequent and intense storms, but lower annual rainfall. This has the potential to increase rainfall intensities for storms leading to increases in the frequency and magnitude of flooding to catchments and waterways in the vicinity of the NWRL project. Climate change is also expected to result in sea level rise however, due to the proximity of the NWRL Project being well away from the ocean this is not expected to have any impact on the Project.

The time period of potential climate change impacts relative to the time period for construction of the EIS 2 works is such that climate change impacts on increased rainfall intensity are not expected to have a significant effect on the construction period. Potential impacts of climate change on the operational phase of the project are discussed below.

Expected trends in rainfall behaviour have the potential to impact on flooding and drainage for the NWRL project in a number of ways. Increased frequency and severity of extreme rainfall events could potentially lead to an incremental increase in:

- flooding of tracks, tunnels, stations, pedestrian underpasses and stabling facilities, foundation instability and damage to associated infrastructure.
- failure of local drainage systems and inundation of station carparks.
- the frequency and extent of scouring at drainage outlets.
- the bypass of water quality systems.

It should be noted that all of these risks already need to be managed under existing conditions. However, climate change has the potential to exacerbate rainfall conditions adding to these risks. It has therefore been necessary to assess the incremental increase in risk due to climate change impacts and identify whether additional allowance needs to be incorporated into the design.

In terms of impacts on local drainage elements, a 10% increase in design rainfall intensities has been adopted in the concept design to provide a nominal allowance for potential climate change impacts. For future project stages it will be important that climate change requirements are captured in design standards to ensure that standards appropriately cater for the potential change in demands on drainage infrastructure. Many of these impacts relate to increased maintenance requirements and more frequent nuisance flooding of drainage systems. Setting of these criteria will require consideration of initial capital cost outlay against risk and impacts as well as the ability to provide for future adaptation.

The potential impacts of climate change on flooding has been assessed in accordance with currently available information and recommended procedures set out in the Floodplain Risk Management Guideline – Practical Considerations of Climate Change (DECC, 2007). The approach adopted to assess and manage the potential impacts of climate change on flooding has involved:

- generally adopting a 10% increase in design rainfall intensities for events up to the 100 year ARI event; but also
- undertaking sensitivity analyses for increases in rainfall intensity of 20% and 30% to identify areas of the Project that may be sensitive to further potential increases in design rainfall intensities.

Potential increases in rainfall intensities have been assessed as part of the hydrologic and hydraulic modelling carried out for the Concept Design development. Results are documented in Appendices B and C. A summary of the potential impacts on 100 year ARI flood levels under the range of scenarios considered is provided in Table 12.

In accordance with current research and best practice, climate change impacts have not been included in PMF assessments. Consequently, where the PMF has been adopted as the design standard (e.g. for tunnel entries and below ground stations) then the design is less susceptible to impacts due to climate change. This is reflected in the outcomes included in Table 12.

Table 12 Potential Climate Change Impacts for 100 year ARI event

Location	Scenario (Increase in Rainfall Intensities)			Comments	Design Outcomes
	+10%	+20%	+30%		
Devlins Creek Tributary	+0.14 to 0.17m	+0.20 to 0.30m	+0.30 to 0.50m	Results show that flood levels are relatively sensitive to increased rainfall intensities, which is largely due to the confined nature of the floodplain.	Entry points to the Epping Services Facility have been designed to the PMF level, which is 2.5m above the 100 year ARI flood level and so potential flood impacts due to climate change are not expected to affect the design.
Cattai Creek	+0.10 to 0.20m	+0.30 to 0.40m	+0.30 to 0.40m	Results show that flood levels are relatively sensitive to increased rainfall intensities, which is largely due to the confined nature of the floodplain.	Showground Station has been located outside the PMF extent, which is over 3m above the 100 year ARI flood level. Consequently, potential flood impacts due to climate change are not expected to affect the design.
Elizabeth Macarthur Creek	+0.02 to 0.15m	+0.07 to 0.30m	+0.10 to 0.45m	Results show that flood levels are relatively sensitive to increased rainfall intensities, which is largely due to the confined nature of the floodplain and local controls such as road crossings.	Between Bella Vista and Kellyville Stations the alignment has been located outside the PMF, which is typically 1m higher than the 100 year ARI flood level. Consequently, potential flood impacts due climate change are not expected to affect the design.

Location	Scenario (Increase in Rainfall Intensities)			Comments	Design Outcomes
	+10%	+20%	+30%		
Caddies Creek	+0.05 to 0.30m	+0.10 to 0.50m	+0.15 to 0.70m	The greatest increases in flood level are experienced upstream of hydraulic controls such as road crossings. In these areas, a 30% increase in rainfall would result in an increase in flood level of up to 0.4m when compared to a 10% rainfall increase. Within the broad floodplain areas flood levels are relatively insensitive to increases in rainfall intensity.	The viaduct has been designed for the 100 year ARI flood with 10% rainfall increase and 0.5m freeboard. Therefore, additional allowance for a 30% rainfall increase is not considered warranted.
Caddies Creek Tributary 5	+0.03 to 0.05m	+0.06 to 0.10m	+0.08 to 0.15m	Results show that flood levels are relatively insensitive to increased rainfall intensities, which is due to the broad nature of the floodplain relative to the magnitude of flow. Increases in flood levels for 30% increase in rainfall are generally less than 0.1m higher than a 10% rainfall increase.	The viaduct has been designed for the 100 year ARI flood with 10% rainfall increase and 0.5m freeboard. Therefore, additional allowance for a 30% increase is not considered warranted.
Caddies Creek Tributary 4	+0.05 to 0.06m	+0.10 to 0.11m	+0.15 to 0.16m	Results show that flood levels are relatively insensitive to increased rainfall intensities, which is due to the broad nature of the floodplain relative to the magnitude of flow. Increases in flood levels for 30% increase in rainfall are generally less than 0.1m higher than a 10% rainfall increase.	The viaduct has been designed for the 100 year ARI flood with 10% rainfall increase and 0.5m freeboard. Therefore, additional allowance for a 30% increase is not considered warranted.

Location	Scenario (Increase in Rainfall Intensities)			Comments	Design Outcomes
	+10%	+20%	+30%		
Second Ponds Creek	+0.05 to 0.08m	+0.11 to 0.16m	+0.15 to 0.28m	Results show that flood levels are relatively insensitive to increased rainfall intensities, which is due to the broad nature of the floodplain relative to the magnitude of flow. Increases in flood levels for 30% increase in rainfall are up to 0.2m higher than a 10% rainfall increase within the project corridor.	The viaduct has been designed for the 100 year ARI flood with 10% rainfall increase and 0.5m freeboard. Therefore, additional allowance for a 30% increase is not considered warranted.
First Ponds Creek	+0.01 to 0.06m	+0.02 to 0.13m	+0.04 to 0.17m	Results show that flood levels are relatively insensitive to increased rainfall intensities, which is due to the broad nature of the floodplain relative to the magnitude of flow. Increases in flood levels for 30% increase in rainfall are typically less than 0.1m higher than a 10% rainfall increase within the Project corridor.	Allowance for the future extension has been designed for the 100 year ARI flood with 10% rainfall increase and 0.5m freeboard. Therefore, additional allowance for a 30% increase is not considered warranted.

5.2 Water Quality

5.2.1 General

The construction and operational phases of the proposal both have the potential to impact on the water quality of the receiving environment. The scale and nature of potential impacts associated with construction works are potentially greater than those under the operational phase.

The operational impacts of the proposal addressed in this EIS 2 include potential changes in the hydrologic regime leading to increased erosion and sedimentation and pollutant generation from the rail infrastructure, station precincts and ancillary facilities.

The construction works would involve excavation in many locations, resulting in disturbance and exposure of the underlying soils. This has the potential to lead to increased erosion and sediment transport and ultimately sedimentation in downstream water bodies. The potential for sediment transport and sedimentation issues is influenced by factors such as severity of storm events, the slope and footprint of disturbed area and the management controls that are implemented on site.

Construction works covered under EIS 2 include the stations and associated precincts, services facilities, stabling facility and other ancillary facilities.

5.2.2 Operational Phase

The increased impervious surfaces associated with the works (such as building roofs and paved areas) have the potential for adverse impacts on the hydrological regime in terms of increased runoff volumes and peak flows. This can lead to a range of impacts including increased erosion and sedimentation. Water Sensitive Urban Design Principles will be incorporated into the design to minimise the impacts of the works on the existing hydrologic regime. Such measures would include:

- Managing total runoff volumes through the use of rainwater tanks at stations and stabling depot buildings and measures that promote stormwater infiltration (such as pervious paving and raingardens).
- Minimising increases in peak flows through the use of detention and retention measures (such as water quality ponds). Stormwater detention is discussed in Section 5.1.5.1.
- Preserving and enhancing the amenity of waterways by providing more natural vegetated measures in lieu of concrete channels.
- Treating stormwater through a range of at source and end point measures that are integrated with the urban landscape. Such measures would include the use of raingardens and bioretention swales to treat runoff from carparks and streets and water quality ponds integrated into public areas.

The station precincts will be areas of high vehicular and pedestrian traffic which has the potential to generate a significant amount of pollutants. Where discharge will be to an existing waterway, there is the risk of an increase in pollutant loads reaching waterways. Runoff will need to be treated prior to discharge into the receiving drainage systems. Water quality treatment measures (including a combination of swales, water quality basins, GTPs) will be provided at outlet points from the drainage system before discharge into existing waterways to mitigate impacts to these waterways.

At the Tallawong Stabling Facility, maintenance activities such as the wash down and general maintenance of trains have the potential to generate considerable volumes of pollutants. However, these activities will be carried out in a covered maintenance building and wash down water will be collected in a separate system for treatment and reuse, thus avoiding any potential for such pollutants to enter the local drainage system.

The potential for pollutant generation along the rail tracks is relatively low and would be mainly relate to sediments (including brake dust particulate matter).

Groundwater seepage into the tunnel will be collected and pumped to the Lady Game Drive Water Treatment Plant (WTP). The treated seepage is then discharged to the Lane Cove River. Refer to Chapter 8 of the main EIS 2 report for further discussion.

The adopted water quality measures will be integrated into a holistic approach to water management that is tailored to the specific requirements of the Project and the potential for pollutant generation. A summary of potential pollutants and proposed measures for each drainage element is provided in Table 13.

Table 13 Summary of Drainage Elements and Associated Water Quality Impacts and Mitigation Measures

Drainage Element	Impacts and Potential Pollutant Generation	Mitigation and Requirements	Examples of Typical Treatment(s)
Track Drainage	Low volumes of pollutant generation are expected from track runoff, and would mainly relate to sediments (including brake dust particulate matter).	Management of pollutants to be accommodated within the design through the implementation of at source measures such as the use of vegetated swales in lieu of concrete or bitumen lining, and absorption trenches with slotted pipes in lieu of unslotted pipes and collection pits. Where unslotted pipes are required (for example at under line crossings) then silt traps would be provided by lowering the invert of the pit 300mm below the downstream pipe outlet. Absorption trenches and vegetated swales reduce flow velocities thereby removing water-carried sediment. This type of arrangement provides natural water filtering which reduces the impacts of runoff pollutants and minimises sediment deposition within the receiving bodies downstream. Treatment of runoff from the viaduct to be incorporated into the broader precinct stormwater management strategy.	Absorption trenches with slotted pipes along tracks in lieu of unslotted pipes where feasible. Grassed swales where room available. Litter baskets on inlets at station platforms.

Drainage Element	Impacts and Potential Pollutant Generation	Mitigation and Requirements	Examples of Typical Treatment(s)
Groundwater from Below Ground Stations and Tunnels	Groundwater flows collected at below ground stations and (nominal) flows collected from the tanked tunnels will potentially be of poor quality, requiring treatment prior to discharge into the surface water system. Other runoff within the tunnels would be from washdown of station platforms, which would be high in nutrients. Litter from the stations could also transported along the tunnels and into the drainage system. Refer to Chapter 8 of the main EIS report.	Groundwater flows and other flows from the below ground stations and tunnels will be collected and pumped to the existing treatment facility at Lady Game Drive. Refer to Chapter 8 of the main EIS report.	Litter guards at station platforms. Central water treatment facility at Lady Game Drive.

Drainage Element	Impacts and Potential Pollutant Generation	Mitigation and Requirements	Examples of Typical Treatment(s)
Stations Precincts	Pollutant generation at stations and carparks is mainly expected to be due to litter, grease and oils from motor vehicles, sediments and wash down chemicals at stations.	Water quality measures at the stations and associated carparking facilities will be provided in accordance with Sustainable Design Guidelines for Stations, Commuter Car Parks and Maintenance Facilities (Transport Construction Authority NSW, 2011) and local Council requirements.	High traffic areas such as station plazas and carparks: • Gross pollutant traps • Filter pits • For roads, footpaths and public space: • Grassed swales • Biorentention trenches and raingardens • Water quality ponds
Stabling Facility	Facility areas and access roads may be subject to grease and oils from motor vehicles, sediments and general gross pollutants. Cleaning of trains would be carried out in a maintenance building. Wash water would be collected in a separate system, thus avoiding wash water from entering the local drainage system.	Water quality measures would be provided in accordance with the NSW Sustainable Guidelines for Rail (TfNSW, 2011). Water quality measures have been incorporated into the conceptual site layout including vegetated swales and a water quality basin at the outlet of the site drainage system. Rainwater would be harvested from the roof of the train maintenance building for reuse on site. An oil and grit separator would be required in the event of stabling any diesel trains used for track maintenance works.	At source measures integrated with the drainage system: • Slotted pipe in filter trench • Grassed swales • At major outlets: • Water quality ponds • Gross pollutant traps and filters.

5.2.3 Construction Phase

The construction of the stations, precincts and services facilities is tied in with the construction of the major civil works, as discussed in the EIS 1. The water quality requirements of the construction sites for the EIS 2 works would be consistent with those required for the major civil construction works as described in EIS 1.

5.2.3.1 Erosion and Sedimentation

Works involving excavation would have the greatest potential to result in sediment transport and sedimentation issues downstream. Such works under the EIS 2 would include:

- general civil works associated with the construction of the rail precincts, temporary and permanent roads and ancillary station facilities,
- handling of spoil associated with the above activities.

These works affect all construction sites in one form or another and pose the greatest risk where they occur near waterways (such as Caddies Creek and its tributaries), on steep slopes or on land subject to overland flow or flooding. A management framework and site specific controls would need to be developed and implemented during the construction phase of the project to reduce the risks of sedimentation in down gradient water bodies due to the proposed constructions works.

Preliminary soil risk maps were prepared and included in EIS 1. These maps identified areas more likely to be prone to erosion due to the construction works.

The preliminary risk mapping was prepared based on soil landscapes, ground conditions (rock or soil), erodibility, slope, extent of clearing required, location of works relative to sensitive receiving environments and the type of construction works being undertaken (piers/piling works, fill earthworks or cut slopes). The soil risk mapping would be further developed as part of the Construction Environmental Management Plan.

Procedural and physical management measures would be implemented during construction to retain sediment at the work locations. Measures could include the use of sediment basins or bunded swales. During significant rainfall events however, there is the potential that these sediment control measures will become completely filled to capacity and surcharge into the downstream environment. In such large events, higher quantities of sediment and pollutants from the site works may be discharged into downstream water bodies, potentially affecting local water quality.

However, providing the site measures to control erosion and sedimentation are designed to an appropriate standard, then the spills would only occur following significant volumes of runoff and the quantity of sediment or pollutant would be appropriately diluted.

5.2.3.2 Spoil Handling

The construction of the NWRL will generate a significant quantity of spoil as a result of excavations for tunnels, below ground stations and services facilities, as well as the above ground civil works. Spoil generation is expected to be greatest from the tunnel excavation, which was assessed in EIS 1.

5.2.3.3 Works in Riparian Areas

Generally the proposed works in riparian areas are associated with the major civil works assessed in EIS 1. This includes construction of viaducts and bridges over the major watercourse crossings of Caddies Creek and tributaries, Elizabeth Macarthur Creek and Second Ponds Creek have the potential to impact upon water quality.

5.2.3.4 Potential for Spills

The release of potentially harmful chemicals and other substances into the environment may occur as a result of the proposed construction works with adverse impacts on the water quality in receiving waters downstream of the project. Such substances would include acids and chemicals from washing processes, construction fuels, oils, lubricants, hydraulic fluids and other chemicals. Release of these substances might occur due to spills, as a result of equipment refuelling, failure and maintenance of machinery, via treatment and curing processes for concrete, as a result of inappropriate storage, handling and use of the substances or from the disturbance and inappropriate handling of contaminated soils.

A management framework will be required to reduce the potential for environmental releases of potentially harmful chemicals and to reduce the risk of any such releases entering local waterways.

5.2.3.5 Demolition Works

The construction works will require the acquisition and demolition of existing buildings within the project extent. Appropriate mitigation measures including stockpiling and management of potentially contaminated material will be required to prevent the possible movement of material into receiving waters.

5.2.3.6 Soil Salinity

Salt occurs naturally within many parts of the Australian landscape. However, urbanisation practices can increase the movement of water through the soil profile and thus exacerbate salinity. Excess salt levels can affect vegetation and building materials such as concrete and steel.

Soil salinity has been identified as a growing problem in the Western Sydney region. Salinity potential maps that identify the potential risk of soil salinity have been prepared by the former Department of Infrastructure, Planning and Natural Resources (DIPNR, 2002). Based on these maps, areas around Caddies Creek, First Ponds Creek and Second Ponds Creeks show high salinity potential or known salinity.

It is recommended that the presence of soil salinity be identified and appropriate mitigation measures adopted in accordance with Western Sydney Regional Organisation of Council's Draft Salinity Code of Practice and the former DIPNR Guidelines to Accompany Map of Salinity Potential in Western (2002).

5.2.3.7 Acid Sulphate Soils

Acid sulphate soils can weaken concrete and steel infrastructure, which can increase maintenance and replacement costs. In addition, sulphuric acid can damage aquatic environments, if allowed to be released during construction.

Acid sulphate soil risk mapping has been undertaken by the former Department of Land and Water Conservation (DLWC). These maps show that the project lies within areas designated as ‘no known risk’ of acid sulphate soil or potential acid sulphate soil (DLWC, 1998). On this basis the potential for impacts associated with acid sulphate soils is expected to be low.

6.0 Mitigation Measures

6.1 Flooding

6.1.1 Operational Phase

The assessment of flood impacts associated with the EIS 2 works has provided an understanding of the scale and nature of flood risks to the project and surrounding environment. This assessment has been used to develop a framework to manage impacts within the concept design and establish criteria for future design development. Key elements of this framework to manage flood impacts would include:

- Entries to below ground stations will be located above the PMF level for mainstream flooding and local measures provided to manage the ingress of runoff from local overland flooding up to the PMP.
- The stabling facility at Tallawong Road would be located above the 100 year ARI flood level.
- Tunnel entries will be located above the PMF level for mainstream flooding and local measures provided to manage the ingress of runoff from local overland flooding up to the PMP.
- The rail line will be located above the 100 year ARI flood level to provide an appropriate level of flood immunity.
- Entries to below ground services facilities would be located above the PMF level for mainstream flooding and local measures provided to manage the ingress of runoff from local overland flooding up to the PMP.
- Critical rail system infrastructure such as substations and sectioning huts will be located a suitable level above the 100 year ARI peak flood level to protect against mainstream and local overland flooding.
- Works within the floodplain will be designed to minimise adverse impacts on adjacent development for flooding up to the 100 year ARI event. Assessment will also be made for regional impacts during flooding in excess of the 100 year ARI event up to the PMF in the context of impacts on critical infrastructure, flood hazards and emergency evacuation in accordance with the NSW Floodplain Development Manual.
- The extent of net filling within the 100 year ARI flood extent will be minimised along the rail embankment and at all precincts.
- Onsite detention systems (OSD) will be provided at the precincts where required to mitigate impacts associated with increased impervious areas and in accordance with local Council requirements.
- Local drainage systems and overland flowpaths at all precincts will be designed to provide appropriate flood immunity to the precincts and minimise the risk of ingress of floodwaters to the underground stations. The flood standard adopted at each precinct will be developed taking into consideration the magnitude of inflows and the potential risks to the project works, facilities and personal safety.
- The works would be designed to manage the potential impacts due to climate change in accordance with the Practical Considerations of Climate Change – Floodplain Risk Management Guideline (DECC, 2007).

6.1.2 Construction Phase

A stormwater management plan that identifies the appropriate design standard for flood mitigation based on the duration of construction, proposed activities and flood risks would be developed for each construction site. The plan would develop procedures to ensure that threats to human safety and damage to infrastructure are not exacerbated during the construction period.

Appropriate mitigation measures to be implemented will vary depending on the nature of the risks and sensitivity of the particular situation but would include consideration of the measures outlined below.

- Temporary diversion or pumping of low flows around the works area.
- Minimising the need or extent of any obstructions required to be placed within the waterway area.
- Programming or staging any construction associated with creek/channel works or the temporary transverse culverts to minimise the total time that works are undertaken in the vicinity of watercourses and thereby minimise the risk exposure.
- To better facilitate construction methods and reduce potential erosion/scour problems, permanent diversion of small channels in localised areas might be considered for situations where the permanent works (such as bridge piers) may be required to remain adjacent to or partially obstructing the waterway.
- Ensuring construction equipment (or excess material) is removed from the waterway or floodplain areas if wet weather is approaching and at the completion of each day’s work activity.
- Strategically placing temporary levees or bunds to contain potential impacts resulting from temporary works in the floodplain and minimise the risk to surrounding properties which might otherwise be affected.

6.2 Water Quality

6.2.1 Operational Phase

Water quality measures for the stations precincts and rail infrastructure will be incorporated into the design of stormwater drainage systems in accordance with:

- NSW Sustainable Guidelines for Rail (TfNSW, 2011),
- Australian Runoff Quality (IEAust, 2006), and
- Relevant Local Council and North West Growth Centre standards.

A holistic approach to water quality and stormwater management would be adopted that incorporates Water Sensitive Urban Design principles to minimise impacts on the existing hydrologic regime. Such measures would include:

- Managing total runoff volumes through the use of rainwater tanks and measures that promote stormwater infiltration.
- Minimising increases in peak flows through the use of detention and retention measures as appropriate.
- Preserving and enhancing the amenity of waterways by maintaining or providing natural vegetated measures.
- Treating stormwater through a range of at source and end point measures that are integrated with the urban landscape.

A surface water quality monitoring program would be developed post construction for the station precincts, services facilities and the stabling depot to monitor water quality upstream and downstream of the works. The monitoring programme would build on the already considerable amount of water quality data available as presented in Section 4.2. Monitoring procedures and criteria would be established in consultation with local councils and relevant government agencies.

6.2.2 Construction Phase

The location of the construction sites for the EIS 2 works would be consistent with the sites required for the major civil construction works as described in EIS 1. The proposed mitigation measures discussed here will therefore be commensurate with those proposed for the major construction sites.

As a general guiding principle for the construction works, water quality mitigation and management measures would be implemented in accordance with the relevant requirements of:

- Managing Urban Stormwater - Soils and Construction Volumes 1 and 2 (often referred to as the “Blue Book” - Landcom, 2004 and 2006)
- NSW Office of Water Guidelines for Controlled Activities
- ANZECC Guidelines for Fresh and Marine Water Quality
- ANZECC Guidelines for Water Quality Monitoring and Reporting
- Water Management Act 2000

The control and mitigation of potential surface water quality impacts during the construction phase would be defined in a Soil and Water Management Plan (SWMP) prepared as part of the overall Construction Environmental Management Plan (CEMP). The SWMP would be developed to incorporate the most appropriate or “best practice” controls and measures in accordance with “The Blue Book” requirements and the Plan would be continually updated to suit the ever changing needs as the project works progress. Due consideration would also be given to the extent of works and situation relative to the sensitivity of the surrounding environment. Typical mitigation measures to be considered or implemented are outlined in the following sections.

6.2.2.1 Implementation and Monitoring

- Employ a qualified soil conservation officer to advise on appropriate controls and to monitor the implementation and maintenance of such measures.
- Engage all site staff through tool box talks or similar with appropriate training on soil and water management practices.
- A surface water quality monitoring program for the construction period would be developed for all construction sites, including those of the major civil works discussed in the EIS 1, to monitor water quality upstream and downstream of the construction areas. The monitoring programme would commence prior to commencement of any construction works and would build on the already considerable amount of water quality data available as presented in Section 4.2.
- Construction period monitoring would be carried out periodically and after rainfall events as part of the assessment of the operation of water quality mitigation measures. Monitoring during the construction phase of the project would examine a range of appropriate indicators in accordance with standard guidelines.

6.2.2.2 Erosion and Sediment Control

- Minimising disturbed areas and re-vegetating or stabilising such areas as soon as practical as the works progress.
- Utilising cleared vegetation for mulching wherever possible to minimise erosion and filter runoff to trap coarse sediments.
- Installation of appropriate erosion control measures such as silt fencing, straw bales, check dams, temporary soil stabilisation, diversion berms or site regrading.
- Divert clean water runoff away from the works or disturbed areas wherever possible.
- Installation of temporary sediment basins as appropriate (refer Section 6.2.2.3 for preliminary sizing).
- Installation of any permanent scour protection measures required for the operational phase as soon as practical.
- Manage the release of concentrated discharges from the sites through provision of outlet scour protection and energy dissipation in accordance with the “Blue Book” requirements.

6.2.2.3 Preliminary Sediment Basin Sizing

- A preliminary assessment has been carried out to determine the potential sediment basin sizing that may be required to control runoff from each of the construction sites, which include the major civil works as discussed in the EIS 1. Calculations are provided in Appendix C and are based conservatively on disturbance of the entire site. The volume required could be offset by the use of alternative measures such as bunded swales, staging of works and stabilisation of areas. On this basis, the estimated volumes (as summarised in Table C1 of Appendix C) provide an indication of the upper bound size of total basin volume required at each location.
- Multiple basins would be employed for the larger sites to control runoff. The exact size and layout of basins would need to be tailored to suit the ever changing form and needs of the construction site as the works progress.

6.2.2.4 Management of Spills

- Provision of bunded areas for storage of hazardous materials such as oils, chemicals and refuelling areas.

6.2.2.5 Demolition works

- Appropriate mitigation measures including stockpiling and management of potentially contaminated material will be required at building demolition sites to prevent possible movement of material into receiving waters.

6.2.2.6 Soil Salinity

- It is recommended that the presence of soil salinity be identified and where applicable appropriate mitigation measures adopted in accordance with Western Sydney Regional Organisation of Council’s Draft Salinity Code of Practice and the former DIPNR Guidelines to Accompany Map of Salinity Potential in Western (2002).

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7.0 Conclusions

This report has documented the detailed assessment of surface water and hydrology issues that has been carried out for EIS 2. This assessment has established existing baseline conditions with respect to the flooding and water quality environment and identified the nature and extent of any potential impacts associated with the proposed works. The assessment has specifically focused on the potential impacts associated with the permanent infrastructure. However, some consideration of the construction of the various project elements (including stations, precincts, stabling facility and services facilities) has been discussed as appropriate. Where impacts have been identified, a range of mitigation measures and requirements have been proposed to ensure such impacts are minimised.

Flooding

Assessment of flood risks to the project and surrounding environment, and development of appropriate flood standards and mitigation measures has been carried out in accordance with the NSW Floodplain Development Manual (2005), the requirements of the environmental approvals process and industry guidelines.

The Project crosses a number of creeks and watercourses and their associated floodplains. Under the current design a range of works are required within these floodplains, including station precinct works. These works have the potential to change flood behaviour and adversely impact on the surrounding environment. It has therefore been necessary to assess the nature and extent of impacts and ensure that measures can be provided to minimise impacts on surrounding development.

The consequences of flooding to the Project works can affect the serviceability of the rail system, cause damage to infrastructure and risk the safety of rail users and staff. Accordingly, flood standards have been established and design measures have been proposed to manage flood risks to key project elements, including tunnel entries, stations, bridges and viaducts, at grade track sections, and ancillary facilities such as the stabling facility, carparks and tunnel service facilities.

Water Quality

The construction and operational phases of the proposal both have the potential to impact on the water quality of the receiving environment. The scale and nature of potential impacts associated with construction works are potentially greater than those under the operational phase.

Potential water quality impacts during the operational phase of the project are associated with the treatment of groundwater seepage in the tunnels, track and viaduct runoff, precinct drainage and stabling facility wash down areas.

The construction works would involve excavation in many locations, resulting in disturbance and exposure of the underlying soils. This has the potential to lead to increased erosion and sediment transport and ultimately sedimentation in downstream water bodies. The potential for sediment transport and sedimentation issues is influenced by factors such as severity of storm events, the slope and footprint of disturbed area and the management controls that are implemented on site.

Water quality requirements for the Project have been identified in accordance with best practice and relevant guidelines/standards.

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Appendix A

Appendix A - Hydrologic Modelling

Appendix A Hydrologic Modelling

A.1. Overview

Hydrologic modelling has been undertaken to quantify runoff behaviour for the catchments traversed by the Project. To meet the particular requirements of the hydrologic assessment for the Project it was necessary to develop a rainfall runoff routing model that was capable of representing changes in flow behaviour as a result of urbanisation, incorporating flow control measures such as detention basins and generating peak flow and hydrograph outputs suitable for use in the hydraulic modelling.

There are a number of rainfall runoff routing models described in Australian Rainfall and Runoff (IEAust, 1987) that would be suitable for the present application, including RORB, XP-RAFTS and the Watershed Bounded Network Model (WBNM). Of these, WBNM was selected for the present study. WBNM has been widely used for similar hydrologic studies in New South Wales. Its application is based on a well researched and widely used set of model parameters that makes it well suited to applications where limited calibration data is available. For the catchments within the North West Growth Centre (covering Caddies Creek, Elizabeth Macarthur Creek, Strangers Creek, Second Ponds Creeks and First Ponds Creek) previous studies have adopted XP-RAFTS. In this respect the use of WBNM for the present study provides a point of comparison and validation of flows estimates for the catchments.

The XP-RAFTS models of previous studies were generally developed for regional flood mapping and planning purposes. The WBNM modelling carried out for the Project has been specifically developed to define flood behaviour in the vicinity of the Project corridor and address a number of particular requirements of the project, including the assessment of:

- both existing and future development scenarios,
- potential impacts due to climate change, and
- quantification of flood behaviour in the PMF event to manage flood risk to critical infrastructure.

WBNM models were established to represent the creek and waterway systems traversed by the Project. Discussion on the model setup, validation and results for these areas is provided in the following sections.

A.2. Model Arrangement

A.2.1 General

WBNM is a rainfall event based hydrologic model that represents the tributaries or flow paths of a catchment as a series of sub-catchment areas based on the watershed boundaries. Sub-catchments are linked together to replicate the rainfall runoff process through the natural or urban stream network. Information is required to define the physical characteristics of the catchment, including:

- Subcatchment size, connectivity and routing,
- Proportion of imperviousness within the catchment (due to urbanisation),
- Rainfall losses due to depression storage and infiltration,

- Details of detention basins and other flow control structures that influence runoff response.

Five hydrologic models were established representing the following catchment areas:

- Devlins Creek Tributary
- Cattai Creek
- Caddies Creek (including Elizabeth Macarthur Creek, Strangers Creek and Tributaries3, 4 and 5)
- First Ponds Creek
- Second Ponds Creek

A.2.1 Model Inputs and Parameters

Catchment Characteristics

Sub-catchment boundaries were delineated using a combination of 0.5m and 2m topographic contours generated from Airborne Laser Survey (ALS). Aerial photography was used to identify urban features such as roads, dams and other man made features influencing flow patterns and catchment delineation. The aerial photography was also used to identify the extent of development and proportion of impervious area. Catchment details are summarised in Table A.1.

For the well established areas in the south east portion of the project (including Devlins Creek Tributary and Cattai Creek) future land use is not likely to have a significant impact on catchment characteristics and runoff behaviour. However, for catchments within the North West Growth Centre (covering Caddies Creek, Elizabeth Macarthur Creek, Strangers Creek, First Ponds Creek and Second Ponds Creek) a considerable degree of development is underway and ongoing. To consider flow behaviour under ultimate conditions a review was made of available masterplanning documentation including proposed water management strategies that include the provision of detention basins to offset potential increases in flow due to urbanisation.

Particular areas within the North West Growth Centre where significant future development has been identified include Balmoral Release area, Area 20 Precinct, Alex Avenue, Riverstone, The Ponds and Beaumont Hills. Many of these areas are all largely undeveloped under existing conditions. As such the effect of adopting future land use values in these areas can have a considerable impact on design peak flows when compared to existing undeveloped conditions.

Within the North West Growth Centre, hydrologic assessment has been undertaken for both existing and ultimate scenarios to ensure that the design is future proofed against ultimate catchment characteristics and runoff behaviour. Future land use and the proportion of impervious area have been determined based on masterplan layouts for the area precincts and associated surface water management plans. Plans reviewed for this study are outlined in the main body of the report. Catchment average impervious area factors adopted for existing and ultimate conditions are summarised in Table A.1.

While significant future development is planned for First Ponds Creek, there is limited information available in studies and plans on the proposed stormwater management layout in order to define the ultimate catchment conditions.

Detention Basins

The catchments of Caddies Creek, Strangers Creek, Elizabeth Macarthur Creek and Second Ponds contain a number of detention basins that have been incorporated into the surface water management strategies for these areas. The objective of these regional basins is to offset increased runoff as a result of the increased urbanisation of these areas. In assessing ultimate catchment conditions it is therefore important that the influence of the basins in controlling runoff is adequately represented in the WBNM model. Details of basin storage and outlet configuration were mainly obtained from the SKM 2009 study.

The SKM 2009 study identified 21 detention basins in the Caddies Creek, Strangers Creek, Elizabeth Macarthur Creek and Second Ponds area. A number of these basins are relatively small with storage capacities of less than 10,000m^3 and designed for local scale detention. For the purposes of the present study only the larger regional detention basins that would influence flow behaviour in the vicinity of the rail corridor have been included in the WBNM model.

An additional basin (referred to as Kellyville Ridge Dam) that was not documented in the SKM 2009 study was identified in the Second Ponds Creek catchment. The basin is located south (upstream) of Schofields Road and drains to Basin 40. Storage details for the basin were based on aerial survey while the outlet configuration was based on a report by Patterson Britton (2005).

In the Second Ponds catchment the following basins were included:

- Basin 40
- Basin 41
- Large Parklea Prison Pond
- Kellyville Ridge Dam

In the Caddies Creek catchment (including Strangers Creek and Elizabeth Macarthur Creek) the following basins were modelled:

- Basin Norwest 1
- Basin Norwest 2
- Basin Norwest 3
- Basin 5
- Basin 13
- Basin 20
- Basin 21
- Basin 35

Table A.1. Catchment Summary

Catchment	Rail Chainage (m)	Catchment Area (Hectares)	Impervious Coverage (%)				Catchment Description
			This study		SKM (2009) Note 1	JWP (2010) Note 2	
			Existing	Ultimate	Ultimate	Ultimate	
Devlins Creek Tributary	25500	230	74	74	N/A	N/A	Joins Devlins Creek at M2 Motorway, approximately 500m downstream of the NWRL corridor. The catchment consists mostly of medium to high density residential areas.
Cattai Creek	36400	288	55	55	N/A	N/A	The catchment consists mainly of medium density residential areas, while the western parts of the catchment cover light industrial and commercial zones.
Strangers Creek	38300	36	34	53	60	N/A	The Strangers Creek catchment to the NWRL corridor consists of residential and commercial development. The creek continues further north of NWRL, joining Caddies Creek some 5km further downstream. A significant proportion of the catchment through Kellyville has not been fully developed at present.
ElizabetH Macarthur Creek	43500	254	30	74	74	N/A	The creek drains part of the Norwest Business Park. The existing catchment consists of a mix of residential, commercial and rural. A significant proportion of Elizabeth Macarthur Creek through Kellyville has not been fully developed at present. A large area in the vicinity of Balmoral Road and Burns Road is currently undeveloped, but with future urbanisation proposed.
Caddies Creek	43400	711	50	63	60	N/A	The catchment of Caddies Creek consists mostly of medium density residential areas, with small pockets of commercial and open space.

Catchment	Rail Chainage (m)	Catchment Area (Hectares)	Impervious Coverage (%)				Catchment Description
			This study		SKM (2009) Note 1	JWP (2010) Note 2	
			Existing	Ultimate	Ultimate	Ultimate	
Caddies Creek Tributary 5	43600	91	50	60	60	N/A	Joins Caddies Creek immediately east of the NWRL corridor. Consists mostly of low to medium density residential areas, with small areas of commercial and open space.
Caddies Creek Tributary 4	44500	89	50	75	60	N/A	Joins Caddies Creek approximately 400m east of the NWRL corridor. Consists mostly of low to medium density residential areas.
Caddies Creek Tributary 3	45200	35	34	37	37	N/A	Joins Caddies Creek approximately 600m east of the NWRL corridor. Drains Castlebrook Cemetery and Windsor Road immediately upstream (west) of the NWRL corridor.
Second Ponds Creek	47000	621	47	61	57	62	The catchment at present consists of both rural and medium density residential areas. Most of the area that is as yet undeveloped has, however, been zoned as residential under the North West Growth Centres planning process.
First Ponds Creek	Immediately west of project limit	300	11	Note 3	N/A	N/A	The catchment at present consists of both rural and medium density residential areas. Most of the area that is as yet undeveloped has, however, been zoned as residential under the North West Growth Centres planning process.

Notes:
1. Rouse Hill Integrated Stormwater Strategy Review – Hydrology, Hydraulics and Water Quality Review (Final Report), SKM, 10 July 2009
2. Area 20 Precinct, Rouse Hill Water Cycle Management Strategy Report Incorporating Water Sensitive Urban Design Techniques, J. Wyndham Prince, October 2010
3. Limited details available on final stormwater management strategy for First Ponds Creek. Existing conditions adopted on the basis that future development will require detention basins to offset increases in peak discharges.

Hydrologic Model Parameters

Ideally, hydrologic model parameters should be calibrated against observed historical events and stream gauging data to provide some measure of confidence in results produced. Within the catchments considered here, there are no flow gauging data available. Consequently, model parameters were adopted based on recommended values in AR&R (IEAust, 1987) for ungauged catchments and past research. Adopted values and model results were validated by comparison with previous studies where data was available (refer to Section A.3).

The following WBNM hydrologic model parameters were adopted for this study:

Lag Parameter ‘C’	= 1.3
Initial Loss	= 5 mm for design events up to 100 year ARI
	= 0 mm/h for PMF event
Continuing Loss	= 2.5 mm/h for design events up to 100 year ARI
	= 0 mm/h for PMF event

While AR&R 1987 recommends a lag parameter ‘C’ of 1.3 for ungauged catchments, based on a number of subsequent studies (including Boyd and Bodhinayake, 2006), the WBNM theory manual now recommends a lag parameter ‘C’ of 1.6 for ungauged catchments. Both values were trialled and 1.3 was found to produce a better comparison with previous study results and alternative methods (i.e. Probabilistic Rational Method).

The initial rainfall loss adopted for events up to the 100 year ARI is lower than values adopted for previous studies, such as the SKM 2009 study for Caddies Creek and the JWP 2010 study for Second Ponds Creek which adopted initial losses of 15 - 25mm. These higher values are considered appropriate for the more natural riparian corridor areas. However, for developed areas initial loss rates would be expected to be lower. Also, larger storm events are generally preceded by smaller rainfall events that tend to fill up available surface storage in the catchment. For these reasons an initial loss of 5mm has been adopted for the present study.

The continuing loss rate adopted for events up to the 100 year ARI is consistent with previous studies.

A.3. Design Rainfall

Design rainfalls and temporal patterns were calculated in WBNM based on the procedures set out in AR&R. Design rainfall coefficients for input to WBNM were obtained from the Bureau of Meteorology, (BOM) website.

Probable Maximum Precipitation (PMP) rainfalls were also calculated in WBNM using input parameters from “The Estimation of Probable Maximum Precipitation in Australia: Generalised Short- Duration Method” (BOM, 2003).

The rainfall patterns vary considerably along the alignment, mainly due to the changes in topography. Consequently, design rainfall Intensity Frequency Duration (IFD) data relevant to each different area were developed and are summarised in Table A.2. Typically the design rainfall for the south eastern portion of the Project is higher than the north west.

Table A.2. Design Rainfall Intensities (mm/hour)

Devlins Creek Tributary							
	Average Recurrence Interval (years)						
Duration	**2**	**5**	**10**	**20**	**50**	**100**	**PMP**
5 mins	113	143	160	183	213	236	-
6 mins	106	134	151	172	201	222	-
10 mins	86.5	110	124	142	165	183	-
20 mins	63.1	80.7	90.8	104	122	135	583
30 mins	51.4	65.8	74.1	85.2	99.6	110	488
1 hour	35.1	45.2	51.1	58.8	69	76.6	357
2 hours	23.4	30.4	34.5	39.8	46.9	52.2	228
3 hours	18.4	24	27.3	31.6	37.2	41.6	169
6 hours	12.1	16	18.3	21.2	25.1	28.1	105
12 hours	8.01	10.6	12.2	14.3	17	19	-
24 hours	5.26	7.04	8.12	9.51	11.4	12.8	-
Cattai Creek							
	Average Recurrence Interval (years)						
Duration	**2**	**5**	**10**	**20**	**50**	**100**	**PMP**
5 mins	103	132	148	170	199	220	-
6 mins	96.7	124	139	160	186	207	-
10 mins	79.1	101	114	130	152	169	-
20 mins	57.5	73.3	82.4	94.4	110	122	559
30 mins	46.7	59.5	66.8	76.6	89.3	98.9	471
1 hour	31.8	40.6	45.7	52.4	61.1	67.7	348
2 hours	21.2	27.2	30.7	35.3	41.3	45.8	222
3 hours	16.7	21.5	24.3	28	32.8	36.4	164
6 hours	11	14.3	16.3	18.8	22.1	24.6	103
12 hours	7.27	9.54	10.9	12.6	14.9	16.7	-
24 hours	4.71	6.24	7.16	8.35	9.92	11.1	-

Caddies Creek (including Tributary 4 and 5), Elizabeth Macarthur Creek and Strangers Creek							
	Average Recurrence Interval (years)						
Duration	2	5	10	20	50	100	PMP
5 mins	102	131	147	169	197	218	-
6 mins	95.6	122	138	158	185	205	-
10 mins	78.2	100	113	129	151	167	-
20 mins	56.9	72.6	81.5	93.5	109	121	500
30 mins	46.2	58.9	66.1	75.8	88.4	97.8	424
1 hour	31.4	40.1	45	51.7	60.3	66.8	319
2 hours	20.9	26.8	30.1	34.6	40.4	44.8	204
3 hours	16.4	21	23.7	27.3	31.9	35.4	151
6 hours	10.8	13.9	15.8	18.2	21.3	23.7	96
12 hours	7.09	9.22	10.5	12.1	14.3	15.9	-
24 hours	4.57	6.03	6.9	8.03	9.53	10.7	-
Caddies Creek Tributary 3, Second Ponds Creek and First Ponds Creek							
	Average Recurrence Interval (years)						
Duration	2	5	10	20	50	100	PMP
5 mins	101	129	145	167	196	217	-
6 mins	94.1	121	136	157	184	204	-
10 mins	76.9	98.6	111	128	150	166	-
20 mins	56	71.6	80.5	92.4	108	120	548
30 mins	45.4	58	65.2	74.9	87.4	97	462
1 hour	30.8	39.4	44.3	50.9	59.5	66	343
2 hours	20.5	26.2	29.5	33.9	39.6	44	218
3 hours	16.1	20.6	23.2	26.6	31.1	34.5	162
6 hours	10.6	13.6	15.3	17.6	20.6	22.8	101
12 hours	6.92	8.93	10.1	11.7	13.7	15.3	-
24 hours	4.42	5.82	6.67	7.76	9.21	10.3	-

A.4. Discussion of Results

A.4.1 Model Validation

In the absence of gauged flow data for model calibration, peak flow estimates for the 100 year ARI event from WBNM have been compared with values obtained from Probabilistic Rational Method calculations and previous studies (where available). A summary of the peak flow comparisons is shown in Table A.3.

The PRM has been developed specifically for application to rural catchments. In urbanised catchments, as is typical for the majority of the Project area, catchment flows would be expected to be greater than those estimated by the PRM. An exception to this is in catchments containing detention basins to reduce peak flows to the pre development case.

A direct comparison against PRM is only applicable for the First Ponds Creek catchment, as this is the only catchment where the majority of land is still undeveloped. At other locations PRM estimates are provided as a relative check on order of magnitude.

The SKM 2009 study, GHD 2010 study and JWP 2010 study all used XP-RAFTS hydrologic models. Limited detail is provided in the GHD 2010 report on model parameters and approach. Alternatively, the SKM and JWP reports provide a relatively thorough outline of model parameters and approach to enable a balanced comparison of results.

In contrast to WBNM, XP-RAFTS model input parameters also include catchment slope and roughness to calculate the catchment response behaviour. Based on past experience XP-RAFTS model results can be quite sensitive to the catchment slope and Manning's roughness values adopted. For the SKM study slopes were based on the catchment average rather than the equal area approach recommended for XP-RAFTS. This would be expected to result in higher estimated slopes and consequently higher peak flow results. This would have greater influence on results in the upper reaches (such as Tributaries 3, 4 and 5 of Caddies Creek).

The JWP study used the SKM model of Second Ponds Creek as a basis and developed the model further based on latest information on basin layouts and areas of imperviousness. The JWP results would therefore be expected to be a better or more current representation of ultimate conditions.

A comparison of results for each catchment is provided below.

Devlins Creek Tributary

Limited published flow information is currently available with which to compare the WBNM peak flow estimates. Given the extent of the development the PRM would be expected to be an underestimate of flows and this is illustrated by the fact that the PRM flow is almost half the WBNM flow estimate. Unpublished flow results from a XP-RAFTS model established for the M2 widening project in 2010, shows less than 2% variation in peak 100 year ARI flow estimate at this location. However, it should be noted that the XP-RAFTS model was developed for the purposes of quantifying flows at the M2 and therefore would not have necessarily been calibrated or verified at this location.

Cattai Creek

In the absence of available data on previous studies the WBNM results have been compared against PRM estimates. As would be expected, the WBNM results are significantly higher than the PRM estimate due to the high degree of development within the catchment and associated area of imperviousness.

Strangers Creek

At the location where Strangers Creek crosses the NWRL corridor (immediately east of the Norwest station), the peak flow estimate compares reasonably well with the SKM 2009 study. The results from the SKM 2009 study are 13% higher than the current study. Note that in the SKM model the subcatchment is delineated to the downstream side of Northwest Boulevard, and therefore would be covering a larger catchment.

Peak flow results at the confluence with Caddies Creek also compare well with the SKM 2009 study, with SKM results 12% higher.

Elizabeth Macarthur Creek

Comparison of peak flow estimates along Elizabeth Macarthur Creek with the SKM study shows good similarity with the WBNM results 1-3% higher.

Caddies Creek (including Tributary 5)

For Caddies Creek the WBNM peak flows compare very well with those adopted in the SKM study, with WBNM flows within -2 to +4% of the SKM 2009 study results.

Caddies Creek Tributary 4

For Caddies Creek Tributary 4 the WBNM peak flows are 6% to 15% lower than the SKM 2009 study results. This could be partly attributed to the catchment slopes adopted in the SKM XP RAFTS model, which are typically 1 to 4% for this catchment. These are relatively high and would be expected to have an influence on the peak flow results.

Caddies Creek Tributary 3

WBNM peak flows for Caddies Creek Tributary 3 compare well with results from the SKM 2009 study, with results at the NWRL showing less than 2% variation.

Second Ponds Creek

WBNM peak flows for Second Ponds Creek are 9-19% higher than results from the SKM 2009 study. However, results compare more favourably with the more recent JWP 2010 study being within -1 to +7%.

First Ponds Creek

The WBNM results are consistently higher than both the PRM estimates and the GHD 2010 study. The GHD report does not explicitly identify which XP-RAFTS nodes correspond to Schofields Road or the proposed alignment of the NWRL. Based on the figures provided in the report the closest nodes were selected. The report also does not allow for a comparison at each node of impervious areas or sub-catchment areas between the WBNM and XP-RAFTS models. The discrepancy in predicted peak flows could be due to the uncertainty in the node selection or differences in catchment characteristics.

Summary

Overall, the above comparison shows a reasonable correlation of current peak flow estimates with previous studies and PRM estimates. Results generally compare well with the SKM 2009 study undertaken for Sydney Water. The greatest difference occurs at Second Ponds Creek where the latest information (as documented in the JWP 2010 study) shows appreciable changes to the catchment resulting in higher peak flow estimates.

Noting the above exceptions, the adopted WBNM parameters and results are considered appropriate for use in this study.

Table A.3. Comparison of Peak Flow Results (m^3/s) - 100 year ARI

Catchment	Location	Catchment Area (Hectares)	This Study			Previous Studies			
			Existing	Ultimate	PRM	SKM, 2009 Note 1	GHD, 2010 Note 2	JWP, 2010 Note 3	Maunsell, 2006Note 4
Devlins Creek Tributary	NWRL	230	70.3	-	37.5	-	-	-	-
Cattai Creek	NWRL	174	42.8	-	27.7	-	-	-	-
	Showground Road	324	82.1	-	45.9	-	-	-	-
Strangers Creek	NWRL	36	10.1	12.2	7.9	13.8	-	-	-
	Confluence with Caddies Creek	702	55.8	57.9	85.2	64.8	-	-	-
Elizabeth Macarthur Creek	Burns Road	177	24.3	30.9	28.1	30.5	-	-	-
	Samantha Riley Drive	221	30.1	38.0	33.1	36.9	-	-	-
	NWRL (confl. with Caddies Creek)	249	35.7	43.5	36.7	41.8	-	-	-
Caddies Creek	NWRL	717	63.6	67.6	86.7	66.3	-	-	58.7
	Windsor Road	1041	99.0	106.2	113	107.4	-	-	-
	Confluence with Strangers Creek	1219	101.0	107.9	125.4	109.8	-	-	-
	Sanctuary Drive	2001	175.5	192.5	186.6	184.5	-	-	-

Catchment	Location	Catchment Area (Hectares)	This Study			Previous Studies			
			Existing	Ultimate	PRM	SKM, 2009 Note 1	GHD, 2010 Note 2	JWP, 2010 Note 3	Maunsell, 2006Note 4
Caddies Creek Tributary 5	NWRL (confl. with Caddies Creek)	48.5	11.1	13.1	10.2	13.4	-	-	16.9
Caddies Creek Tributary 4	NWRL	89	20.2	22.6	17.1	24.0	-	-	-
	Confluence with Caddies Creek	142	31.0	34.0	24.1	39.0	-	-	34.9
Caddies Creek Tributary 3	NWRL	34.7	11.3	12.2	7.7	12	-	-	7.4
	Confluence with Caddies Creek	69.7	15.8	16.6	13.9	13.8	-	-	-
Second Ponds Creek	Schofields Road	571	64.2	67.6	71.3	62.0	-	68.3	-
	NWRL	621	74.5	78.8	84.5	66.4	-	73.6	-
First Ponds Creek	Schofields Road	208	36.8	-	31.5	-	32.0	-	-
	NWRL	299	53.6	-	43.0	-	40.0	-	-

Notes:
1. Rouse Hill Integrated Stormwater Strategy Review – Hydrology, Hydraulics and Water Quality Review (Final Report), SKM, 10 July 2009
2. Area 20 Precinct, Rouse Hill Water Cycle Management Strategy Report Incorporating Water Sensitive Urban Design Techniques, J. Wyndham Prince, October 2010
3. Report for Riverstone and Alex Avenue Precincts Post Exhibition Flooding and Water Cycle Management (incl. Climate Change impact on Flooding), GHD, May 2010
4. North West Transitway Section 5: Drainage Southern End Final Design Revision 03 (28 September 2005)/ Drainage Northern End Final Design Revision 03 (13 December 2005), Maunsell

A.4.2 Existing and Ultimate Conditions

Hydrologic modelling has been carried out for both existing and ultimate catchment conditions with peak flow results summarised in Table A.3 for the 100 year ARI event. These results have been reviewed, together with future planning to determine the most appropriate model arrangement to adopt for the design case.

For **Devlins Creek tributary** and **Cattai Creek**, the catchments are well established and there is unlikely to be a significant difference in flow behaviour between existing and ultimate conditions. Future development around Showground Station (in Cattai Creek catchment) is identified in the masterplan for the station precinct. However, this development is not likely to have a significant effect on flows at the rail corridor due to its location and the extent of development relative to existing conditions. For these reasons, existing catchment conditions have been adopted/retained for the design case at these locations.

For **Caddies Creek, Elizabeth Creek, Strangers Creek** and **Second Ponds Creek** the results show that peak flows under ultimate conditions are expected to be equal to or greater than existing conditions. This is to be expected considering that under existing conditions a number of detention basins have already been constructed to compensate for future development. On this basis, ultimate conditions have been adopted for the design case to ensure that the Project is future proofed against increases in catchment runoff once the catchment is fully developed.

As noted previously, based on currently available information there is limited details on the development and stormwater strategy for **First Ponds Creek**. The GHD 2010 study outlined a concept stormwater management strategy to manage peak flows under ultimate development conditions. Results provided in the report show that under ultimate conditions there would be a slight reduction in peak flows at the NWRL corridor in comparison to existing conditions. This is in keeping with development controls for Blacktown City Council (BCC, 2006) and the North West Growth Centres (NSW Department of Planning, 2010) which stipulate that “the developed 100 year ARI peak flow is to be reduced to predevelopment flows through the incorporation of stormwater detention and management devices.”

However, the GHD 2010 study does not provide specific details of proposed basin arrangements and development of the First Ponds catchment is less progressed than other areas of the North West Growth Centre. Therefore, on the basis of currently available information, existing catchment conditions have been adopted for First Ponds Creek for the design case.

A.5. Modelling Results

A.5.1 Design Peak Flows

The WBNM models have been run for both the 100 year ARI and PMF events for a range of storm durations (from 10 minutes to 12 hours for the 100 year, and 15 minutes to 6 hours for the PMF). The critical duration for all the catchments was 2 hours for the 100 year ARI and 45 minutes to 1 hour for the PMF. A summary of estimated design peak flows at the key locations are summarised in Table A.4.

Table A.4. Design Peak Flows (m^3/s) at Key Locations

Location	100 year ARI	PMF
Devlins Creek Tributary	70	304
Cattai Creek	46	230
Elizabeth Macarthur Creek	31	152
Caddies Creek	106	929
Caddies Creek Tributary 5	13	50
Caddies Creek Tributary 4	34	151
Caddies Creek Tributary 3	17	94
Second Ponds Creek	79	692
First Ponds Creek	54	307

A.5.2 Climate Change

Discussion on potential climate change impacts is provided in the main body of the report. As has been identified, climate change has the potential to cause an increase in the frequency and magnitude or severity of storm events. Consequently, in accordance with the DECCW Guideline on Practical Considerations of Climate Change (2007), the following climate change scenarios have been considered for their potential impact on flow behaviour.

Increase in peak rainfall and storm volume:

- Low level rainfall increase – 10%
- Medium level rainfall increase – 20%
- High level rainfall increase – 30%

The results of this sensitivity analysis are summarised in Table A.5. Impacts on peak flood levels are described in Appendix B.

Table A.5. Potential Climate Change Impacts on 100 year ARI Peak Flows (m^3/s)

Location	Rainfall Scenario			
	Base Condition	+10%	+20%	+30%
Devlins Creek Tributary	70	78	86	94
Cattai Creek	46	52	57	63
Elizabeth Macarthur Creek	31	35	38	42
Caddies Creek	106	115	124	140
Caddies Creek Tributary 5	13	15	16	18
Caddies Creek Tributary 4	34	38	42	46
Caddies Creek Tributary 3	17	19	21	22
Second Ponds Creek	79	88	97	106
First Ponds Creek	54	61	67	74

Potential Impacts on Peak Flood Flows

As discussed in the preceding section, sensitivity analyses were carried out for increases in rainfall intensities of 10, 20 and 30%. The corresponding peak flow rates are generally 12%, 23% and 36% greater than the base conditions peak flow rates (refer Table A.6). Caddies Creek and Second Ponds Creek are slightly different, as the detention basins in these two catchments affect the estimated peak flow rates. With increasing rainfall intensities, the flows from the basins change and the timing of peak flows can shift in response so that peak flows from different areas of the catchment now coincide, where they didn't before, thus leading to increased peak flows or vice versa.

Table A.6. Potential Climate Change Relative Impacts for 100 year ARI Peak Flows

Location	Scenario (Increase in Rainfall Intensities)		
	+10%	+20%	+30%
Devlins Creek Tributary	12%	23%	34%
Cattai Creek	12%	24%	36%
Elizabeth Macarthur Creek	12%	24%	36%
Caddies Creek	10%	25%	42%
Caddies Creek Tributary 5	11%	23%	34%
Caddies Creek Tributary 4	11%	23%	34%
Caddies Creek Tributary 3	12%	23%	35%
Second Ponds Creek	16%	32%	47%
First Ponds Creek	12%	25%	38%

Appendix B

Appendix B – Hydraulic Modelling

Appendix B Hydraulic Modelling

B.1. Overview

Hydraulic modelling has been undertaken to determine flood behaviour in the floodplains crossed by the proposed alignment in order to assess flood risks and set minimum design flood levels.

Previous hydraulic modelling studies undertaken for creeks affected by or in the vicinity of the alignment have generally involved one-dimensional models, such as HEC-RAS and MIKE 11. For the more simple and confined waterways of Cattai Creek, Caddies Creek Tributary 3, Second Ponds Creek and First Ponds Creek, a one-dimensional modelling approach is considered appropriate. HEC-RAS models for these waterways in the vicinity of the rail corridor have therefore been developed.

Where the proposed rail alignment traverses the floodplain covering the confluence of Elizabeth Macarthur Creek, Caddies Creek and Strangers Creek, the flood behaviour associated with the widespread flooding and interaction of several creek lines is more complex than at the other creek crossings. Considering also the proximity of the proposed alignment to Windsor Road, the location of tunnel portals in the floodplain and the associated potential flood risks it was deemed appropriate to use a more sophisticated 2-dimensional (2D) hydraulic model of the area using the TUFLOW hydrodynamic software.

The HEC-RAS models developed for previous studies were generally developed for regional flood mapping and planning purposes. The hydraulic modelling carried out for this Project has been specifically developed to define flood behaviour in the vicinity of the project corridor and address a number of particular requirements of the Project, including the assessment of:

- Existing and future development scenarios;
- Potential impacts due to climate change;
- Alternative design options/configurations (vertical and horizontal alignments and waterway structure requirements); and
- Quantification of flood behaviour in the PMF event to manage flood risk to critical infrastructure.

Hydraulic models were established to represent the creek and waterway systems crossed by the Project. Discussion on the model setup, validation and results is provided in the following sections.

B.2. Model Arrangement

B.2.1 General

HEC-RAS (version 4.1.0, January 2010) is able to compute hydraulic characteristics such as estimated water surface profiles and flow velocities at specific locations along a channel using steady or unsteady flow conditions.

TUFLOW (version 2011-09-AB-w32) simulates depth averaged free-surface flows and is specifically orientated towards representing complex flow patterns essentially 2D in nature across floodplain areas while dynamically linked to 1D elements such as defined channels and hydraulic structures.

Six hydraulic models were established representing the following areas:

- Devlins Creek Tributary (HEC-RAS)
- Cattai Creek (HEC-RAS)
- Elizabeth Macarthur Creek, Caddies Creek (including Tributaries 4 and 5) and Strangers Creek (TUFLOW)
- Caddies Creek Tributary 3 (HEC-RAS)
- Second Ponds Creek (HEC-RAS)
- First Ponds Creek (HEC-RAS)

The HEC-RAS cross-sections and TUFLOW digital terrain model (DTM) were based on aerial laser survey data (ALS), supplemented with detailed topographical and hydrometric survey data where available.

Additional field and aerial survey data has been obtained to better define creek lines and hydraulic structures that influence flood behaviour in the vicinity of the Project. Particular areas where field survey has been collected include:

- Cattai Creek
- Elizabeth Macarthur Creek
- Caddies Creek
- Caddies Creek Tributary 3
- Second Ponds Creek
- First Ponds Creek

B.2.2 Boundary Conditions

Depending on the availability of data, the hydraulic models were run assuming either a normal depth or rating curve based on stage-discharge characteristics taken from the SKM model. Adopted tailwater conditions for the different models are summarised in Table B.1.

Table B.1. Adopted Tailwater Conditions

Location	Tailwater Condition
Devlins Creek Tributary	Normal Depth
Cattai Creek	Normal Depth
Caddies , Elizabeth Macarthur and Strangers Creeks	Normal Depth
Caddies Creek Tributary 3	Normal Depth
Second Ponds Creek	Rating Curve
First Ponds Creek	Normal Depth

B.2.3 Bridge Loss Coefficients

HEC-RAS

HEC-RAS has several alternative methods for calculating head losses due to bridge structures:

- Low Flow - four alternative methods that are applicable when the water level is below the underside of the bridge, with the option to let HEC-RAS pick the highest energy answer:
  - Energy
  - Momentum
  - Yarnell
  - WSPRO
- High Flow - two methods that are applicable when the water level is at or above the underside of the bridge:
  - Energy
  - Pressure and/or Weir

The SKM study adopted the highest of either the Momentum or Yarnell approach for modelling low flow at bridges, which was deemed appropriate at the time for modelling bridges with piers. As part of this study, an independent check of potential energy losses around piers was carried out, based on the procedure outlined in the FHWA publication "Hydraulics of Bridge Waterways" (FHWA, 1978) for the proposed rail bridge over Second Ponds Creek.

The losses that HEC-RAS calculates when specifying the Momentum/Yarnell approach could not be validated using the method as outlined in the FHWA document. The HEC-RAS Momentum/Yarnell approach calculates a loss of approximately 1m across the proposed bridge, whereas the FHWA procedure yields a potential energy loss of less than 10mm. The proposed bridge abutments at Second Ponds Creek have been placed outside the 100 year ARI flood extent, and the bridge deck well above the 100 year ARI flood level. As such only the piers would present any obstruction to flow during the 100 year ARI event. The HEC-RAS losses seem excessive, considering that the piers take up less than 10% of the waterway area and the flow velocities would be less than 1m/s in the 100 year ARI event.

It was therefore decided to adopt the Energy approach within HEC-RAS as this is consistent with FHWA and produces more realistic losses across the bridges.

TUFLOW – Road Bridges

For the waterway crossings at Windsor Road and Sanctuary Drive, bridge energy loss coefficients were calculated based on the procedure in the FHWA publication as recommended in the TUFLOW modelling manual. This is approach is also recommend in the Austroads Waterway Design Guide (1994).

TUFLOW – Viaduct

The concept design consists of a viaduct crossing the floodplain of Elizabeth Macarthur and Caddies Creeks between Kellyville Station and Rouse Hill Station. The proposed alignment in this area runs mostly parallel to the general flow direction, unlike at the existing road crossings where the flow is typically more perpendicular. The viaduct piers do, however, still present an obstruction to flow on the floodplain.

The TUFLOW manual provides limited guidance on the best representation and parameter selection of energy losses associated with piers only. Recently published technical papers such as Vienot *et al* (2011) and Leister and Jempson (2011), have explored the representation of bridge piers in TUFLOW in more detail. While some conclusions can be drawn from their work, there is still some uncertainty in the proper application of loss coefficients (as derived from the FHWA (1978) publication) for piers that may not be perpendicular to the general flow direction. Leister and Jempson recommended that until further conclusive research is undertaken, the most appropriate way to model piers within a 2D model is to apply a loss coefficient across the whole waterway cross-section, rather than individual grid cell elements.

A sensitivity analysis, exploring three different representations of loss coefficients was therefore carried out for the viaduct piers in TUFLOW:

- Loss coefficients applied to the whole cross-section: FC (Flow Constriction) lines perpendicular to the general flow direction at the location of each viaduct pier.
- Loss coefficients applied to the whole cross-section: One FC line parallel to the viaduct portion of the route alignment.
- Loss coefficients applied to individual grid cell elements: Discrete FC points representing each viaduct pier.

The loss coefficients were calculated based on the procedure set out in the FHWA publication, taking into consideration the pier width, number of piers and floodplain width for the different configurations. Combined with all three FC representations, the cells representing the individual pier locations were also blocked out.

The results have shown that there is no significant difference in the maximum increase in water levels produced by the three different FC representations. There is, however, some variation in the extent of the impacted areas. The greatest impacts were produced with the perpendicular FC lines, while there was little difference between the other two scenarios.

Taking into consideration the results from the sensitivity analysis and the recommendations made by Leister and Jempson, it was decided to adopt the approach where loss coefficients are applied to the whole cross-section, perpendicular to the general flow direction.

B.2.4 HEC-RAS Model Inputs, Parameters and Setup

HEC-RAS hydraulic model parameters include Manning’s ‘n’ roughness values, expansion and contraction coefficients and culvert entrance and exit loss coefficients. Manning’s ‘n’ values typically in the range of 0.05 to 0.150 (see Table B.2) were adopted for floodplain areas and natural watercourses.

These were derived from site inspection, review of available aerial photography and by reference to recognised texts (refer for example, AR&R (1987) and Chow (1959)).

Hydraulic model expansion and contraction coefficients of 0.1 and 0.3 respectively were adopted, except in the vicinity of bridges and culverts, where they were increased to 0.3 and 0.5 respectively.

Peak discharges for the individual creeks were extracted from the WBNM hydrologic models (refer Appendix A) based on the critical durations at the NWRL. The HEC-RAS models were all run in steady state mode.

Table B.2. HEC-RAS Typical Mannings' n-Values

Land Cover	Mannings' n-Value
Roads	0.025
Waterbodies	0.035
Natural waterway	0.05 – 0.08
Floodplain – sparse vegetation	0.05 – 0.06
Floodplain – dense vegetation	0.08 – 0.1
Floodplain – urban	0.15 – 0.25

A number of culverts and bridges were included in the HEC-RAS models (refer Table B.3). The details for each structure were based on survey data where available, and where this was unavailable details were taken from the hydraulic models developed for previous studies, in particular the Rouse Hill study (SKM, 2009).

Table B.3. Culverts and Bridges Modelled in HEC-RAS

Creek	Location	Existing	Proposed
Devlins Creek Tributary	Beecroft Road	3x1800x1850mm RCBC	-
Cattai Creek	Carrington Road	2x2400x2100mm RCP	-
	20 Anella Avenue	1x2900x2100mm RCBC	-
	Showground Road	1x8.5m bridge	-
Caddies Creek Tributary 3	Windsor Road	2x2400x1200mm RCBC	-
Second Ponds Creek	Schofield Road	5x900mm RCP	-
	NWRL	-	8x22m span bridge Note 1
First Ponds Creek	Schofields Road	2x600mm RCP and 2x1050mm RCP	-
	NWRL	-	-

Note 1: Based on the latest concept design available at the time

Devlins Creek Tributary – Near Epping Station

The proposed NWRL alignment runs below ground between the Epping and Bella Vista Stations. Tunnel vent stacks are proposed just north of the Epping Station alongside Beecroft Road. The proposed vent stacks would be located near a tributary to Devlins Ceek and could lead to flood level impacts to surrounding development if adversely located within the 100 year ARI flood extent.

The creek consists of a man-made channel between Ray Road and Beecroft Road with heavy vegetation cover along the banks. Two drainage lines cross Ray Road from the west and south combining into a single channel just downstream of Ray Road.

A HEC-RAS model, extending from just downstream of Ray Road to 20m downstream of Beecroft Road was developed. The culvert under Beecroft Road has been included in the model. Cross-sections based on ALS data were located at approximately 50m intervals and supplemented with additional survey data obtained in November 2011 specifically for this study.

Cattai Creek – Showground Station

The proposed design calls for a station with associated tunnelling excavated well below surface level. However, the alignment design will need to prevent the ingress of floodwaters into the station surface entrances.

The upper reaches of Cattai Creek in the vicinity of the alignment pass through an urbanised environment but are heavily vegetated and well confined. These factors combined have the potential to generate large debris loads (such as trees, cars, shopping trolleys and other floatable items which may be washed from upstream) and consequently any structures placed within the banks are likely to have blockage issues during flood events.

A HEC-RAS model, extending from approximately 300m upstream of Carrington Road to 300m downstream of Showground Road was developed. Cross-sections based on ALS data were located at approximately 50m intervals. The three culverts under Carrington Road, Showground Road and the property at 20 Anella Avenue have been included in the model.

Caddies Creek Tributary 3 – Rouse Hill Station

The proposed design consists of a viaduct and elevated station at Rouse Hill. There is no proposed embankment in this area, therefore only the viaduct piers and any works associated with the station precinct have the potential to cause flood impacts on adjoin properties.

The creek is relatively confined, especially downstream of Windsor Road. Upstream of Windsor Road a small pond with an earth bund detains flows before they enter the culvert.

A HEC-RAS model, extending from approximately 150m upstream to 450m downstream of Windsor Road was developed. Cross-sections based on ALS data (supplemented by detailed topographic survey data where available) were located at approximately 50m intervals. The culvert under Windsor Road has been included in the model.

Second Ponds Creek – Waterway Crossing near Cudgegong Road Station

The proposed design comprises an embankment and bridge crossing over Second Ponds Creek. The embankment has the potential to cause flood impacts on neighbouring properties and the bridge opening will need to be designed to minimise these impacts.

A HEC-RAS model, extending from approximately 350m upstream of Schofields Road to just downstream of Rouse Road was developed. Cross-sections based on ALS data were located at approximately 50m intervals. The bridge crossings at Schofields Road and the proposed NWRL have been modelled.

First Ponds Creek –Stabling Facility

The proposed design comprises a stabling facility at Tallawong Road with a turnback extending towards First Ponds Creek. The embankment required for the turnback (and future extension of the NWRL to the Richmond line) has the potential to cause flood impacts on neighbouring properties. The design of the vertical alignment for the stabling facility needs to provision for the future rail extension.

A HEC-RAS model, extending from Schofields Road to Gordon Road was developed. Cross-sections based on ALS data and supplemented with ground survey data were located at approximately 50m intervals. The two culverts under Schofields Road are included in the model.

B.2.5 TUFLOW Model Inputs, Parameters and Setup

The proposed rail alignment in the area north of Celebration Drive runs parallel and in close proximity to Elizabeth Macarthur Creek. The flood risks associated with the presence of the creek, as well as general surface drainage to the creek, will have a direct influence on the location of the tunnel portals and/or the details of the alignment structure in this area.

The model extends from about Celebration Drive at Bella Vista to downstream of Sanctuary Drive at Rouse Hill, covering the majority of the Elizabeth Macarthur Creek floodplain, parts of Tributary Creeks 4 and 5, as well the confluence of Caddies, Elizabeth Macarthur and Strangers Creeks. The digital terrain model (DTM) was based on ALS data and detailed field survey of the rail corridor with a grid cell size of 3m.

TUFLOW hydraulic model parameters include Manning's 'n' values which are a friction factor or roughness coefficient affecting the hydraulic efficiency of waterway areas. Manning's n-values were derived from site inspections, review of available aerial photography and by reference to recognised texts and are summarised in Table . For culvert structures, an entrance loss coefficient of 0.5 and a Manning's 'n' value of 0.012 were adopted.

Table B.4. TUFLOW Typical Mannings' n-Values

Land Cover	Mannings' n-Value
Roads	0.025
Waterbodies	0.035
Open spaces (mainly grass)	0.045
Engineered waterway	0.035
Natural waterway	0.05
Floodplain – sparse vegetation	0.05
Floodplain – moderate vegetation	0.06
Floodplain – dense vegetation	0.08
Floodplain – urban	0.15
Buldings	10

A rating curve based on the local hydraulic gradient was specified at the downstream boundary. Flow hydrographs based on the critical duration at the rail alignment were extracted from the WBNM hydrologic model (refer Appendix A) to provide inputs to the hydraulic model.

A number of culverts and bridges were included in the TUFLOW model (refer Table B.5). The details for each structure were based on survey data where available, and where this was unavailable details were taken from the hydraulic models developed for previous studies, in particular the Rouse Hill study (SKM, 2009).

Table B.5. Culverts and Bridges Modelled in TUFLOW

Creek	Location	Existing Infrastructure
Elizabeth Macarthur Creek	Balmoral Rd	2x1500mm RCP 1x2100x1500mm RCBC
	Burns Rd	3x3000x950mm RCBC
	Samantha Riley Drive	1x2700x2400mm RCBC 2x2600x1800mm RCBC 1x2400x800mm RCBC
Caddies Creek	Newbury Drive	16x2700x1800mm RCBC
	Old Windsor Rd	3x2400x1200mm RCBC 2x2200x1400mm RCBC
	Transitway	4x4300x2650mm RCBC
	Windsor Rd	2x18m, 1x15m span bridge
	Sanctuary Drive	2x14m, 1x18m span bridge
Tributary 5	Old Windsor Rd	6x2450x750mm RCBC

Creek	Location	Existing Infrastructure
Tributary 4	Windsor Rd	1x3600x1200mm RCBC 1x2000x1500mm RCBC 1x1400x1200mm RCBC 1x3600x1200mm RCBC 1x3800x1400mm RCBC
Local drainage	Windsor Rd (south of Merriville Road)	2x525mm RCP
Local drainage	Windsor Rd (south of Merriville Road)	4x675mm RCP

B.3. Discussion of Model Results

B.3.1 Model Validation

There are no historical flood data available for the study area for the purposes of model calibration/validation. In the absence of gauged or historical flood level information, the peak flood levels for the 100 year ARI event were compared to previous study results including the SKM 2009, GHD 2010 and JWP 2010 studies. A summary of the peak flood level comparisons is shown in Table B.6.

Limited detail is provided in the GHD 2010 report on model parameters and approach. In comparison the SKM and JWP reports provide a relatively thorough outline of model parameters and approach to enable a balanced comparison of results. These three studies only cover the waterways between the Norwest Business Park and the stabling facility at Tallawong Road. No previous studies are available for the Devlins Creek Tributary and Cattai Creek.

There were several difficulties in comparing peak flood levels between the different studies:

- The SKM study considers only rural and ultimate development scenarios and does not include any allowance for the NWRL. Cross-sections are mostly based on ALS data from 2006/2007 which lack some of the channel definition detail that is evident in the more recent ALS data used in this study.
- The JWP study considers existing and ultimate development conditions for the Area 20 precinct. It used the SKM HEC-RAS model as a basis, but identified some deficiencies in the SKM model within The Ponds development area. The report states that the model of Second Ponds Creek was updated in this area, but the exact details of the changes are unknown. The results reported in the JWP study also make allowance for a bridge over Second Ponds Creek for the future rail alignment but again no details are provided on the assumed bridge configuration.
- The GHD study does report peak flood levels for the MIKE 11 model they developed for First Ponds Creek. However, there is nothing in the report to indicate where the model cross-sections are located, and it is therefore not possible to properly compare peak flood levels for First Ponds Creek against the GHD model.

- The previous hydraulic modelling for Elizabeth Macarthur Creek, Caddies Creek and Strangers Creek was carried out using HEC-RAS (1D) and models were run in steady state mode. For this project TUFLOW (2D) was used to model this area, and the model was run in unsteady mode. As such, because of the different approaches, flood levels in the TUFLOW model are likely to be lower than the HEC-RAS model results for similar peak flows.

HEC_RAS Model Validation

<u>Devlins Creek Tributary and Cattai Creek</u>

No previous hydraulic studies have been undertaken for either of these two waterways. The HEC-RAS models developed for these waterways in this study can therefore not be validated. However, a number of sensitivity runs were undertaken to determine the variation of results to changes in hydraulic roughness and boundary conditions.

<u>Caddies Creek Tributary 3</u>

Previous hydraulic studies that have been undertaken for this waterway include the T-way project (Maunsel, 2005) and the Rouse Hill Integrated Stormwater Strategy (SKM, 2009). The HEC-RAS model developed for the Rouse Hill study does not extend upstream of Windsor Road, so levels in that area can only be compared to the Transitway study.

Modelled flood levels compare very well with the Transitway study (typically within 0.1m difference), but are considerably higher than the SKM Study downstream of Windsor Road. The main reason for this is that peak flows just downstream of Windsor Road in the SKM model are much lower than those adopted in both the Transitway study and the current study. It would appear that the SKM study assumed a sub-catchment draining to Windsor Road that is less than that adopted by the Transitway study and the current study.

<u>Second Ponds Creek</u>

The modelled peak flood levels along Second Ponds Creek between Schofields Road and Rouse Road are approximately 0.2m lower than the JWP model. This is most probably due to the better channel definition used in this study compared to JWP as well as the higher roughness values used in the JWP study (0.09 for channel and 0.105 for overbank areas throughout the Area 20 precinct). A roughness value of 0.09 is considered to be very conservative for a re-vegetated waterway.

The current model compares reasonably well against the SKM model, with flood levels generally in close agreement between the two. While the flows in the SKM model are lower than those adopted in this study, the channel is poorly defined in several locations in the SKM model (i.e. reduced channel capacity) which would lead to higher flood levels.

<u>First Ponds Creek</u>

The GHD report developed for the Riverstone Precinct as part of the North West Growth Area does not provide sufficient information on the location of reported peak flood levels to carry out a valid comparison. The HEC-RAS model for First Ponds Creek developed for this study can therefore not be properly validated.

TUFLOW Model Validation

Elizabeth Macarthur Creek

The modelled peak flood levels along Elizabeth Macarthur Creek between Celebration Drive and Samantha Riley Drive compare reasonably well with the HEC-RAS model results, with flood levels up to 0.4m higher than the SKM study at Celebration Drive. The differences between the two models reduces towards the confluence with Caddies Creek, and the TUFLOW flood level at Samantha Riley Drive is approximately 0.1m lower than the HEC-RAS results.

There are two possible reasons for higher TULFOW flood levels in the upper reaches of Elizabeth Macarthur Creek. The first is that the HEC-RAS model generally has lower flows than those used in the TUFLOW model. The second is that in the HEC-RAS model the road profiles at waterway crossings were generally modelled at a constant level, representing the lowest point along the road. The roads that cross Elizabeth Macarthur Creek generally slope down towards the crossing and then back up again, meaning that the overflow width is generally smaller than what is modelled in HEC-RAS and hence for the same flow across the road, flood levels would be higher. At Samantha Riley Drive, where the two flood levels are very similar, there is also little difference in peak flows and the road does not overtop. This shows that apart from the differences mentioned above, the two models compare reasonably well.

Caddies Creek

Results at Old Windsor Road are very similar between the two models with a difference of -0.05 to +0.05m between the TUFLOW and HEC-RAS models along Tributary 4 and up to +0.1m along Caddies itself. The higher flood level in the TUFLOW model at Caddies Creek is most probably due to the fact that the outlet from Basin 5 is affected by the tailwater levels which are higher in the TUFLOW model than they are in the HEC-RAS model.

Between Old Windsor Road and Windsor Road flood levels in the TUFLOW model are up to 0.4m higher than in the HEC-RAS model. The HEC-RAS model uses the higher of the Momentum or Yarnell approach to estimate the head losses across the bridge structure. This approach only yields an energy loss of 0.15m. This seems very low, considering that the Windsor Road bridge and approach embankment form a considerable and abrupt contraction to flows. In the 100 year ARI event, the flow width along Caddies Creek just upstream of the bridge is approximately 110m, whereas the bridge opening is only 35m. As a comparison, at the Sanctuary Drive bridge, the transition is much more gradual, but the head losses estimated by HEC-RAS are 0.5m across this bridge.

Under these conditions it is considered to be more appropriate to use the Energy approach to estimate the losses across the bridge at Windsor Road. Rerunning the HEC-RAS model using the Energy approach yields head losses of over 0.3m, which means that the TUFLOW modelled flood levels are only 0.1m higher than the HEC-RAS results.

The flow behaviour along Caddies Creek downstream of Old Windsor Road is essentially 2D in nature and the TUFLOW modelling approach is considered to yield more realistic and appropriate results.

Just downstream of Windsor Road flood levels compare reasonably well between the two models with the TUFLOW flood level approximately up to 0.1m higher than the HEC-RAS results. Further downstream towards Sanctuary Drive this situation changes with HEC-RAS flood levels up to 0.5m higher than TUFLOW flood levels. The reason for this is the Momentum approach in the HEC-RAS model used for modelling the bridge at Sanctuary Drive which produces very high losses across the bridge. As discussed in Section B.2.3 this approach may not be very realistic considering the relatively low flow velocities at Sanctuary Drive (less than 1.5m/s).

Noting the above exceptions, the adopted HEC-RAS and TUFLOW parameters and results are considered appropriate for use in this study.

Table B.6. Comparison of Peak 100 year ARI Flood Levels (mAHD)

Catchment	Location	Model Software	This Study	SKM, 2009 Note 1	JWP, 2010 Note 3	Maunsell, 2005 Note 3
Devlins Creek Tributary	NWRL	HEC-RAS	76.0	No comparison possible		
Cattai Creek	NWRL	HEC-RAS	82.4	No comparison possible		
Strangers Creek	Confluence with Caddies Creek	TUFLOW	39.8	40.0	-	-
Elizabeth Macarthur Creek	Burns Road	TUFLOW	56.5	56.3	-	-
	Samantha Riley Drive		47.8	47.9	-	-
	NWRL (confluence with Caddies Creek)		44.3	44.3	-	-
Caddies Creek	Old Windsor Road	TUFLOW	46.4	46.3	-	46.3
	NWRL		43.6	43.3	-	-
	Windsor Road		43.1	42.8	-	-
	Confluence with Strangers Creek		40.0	40.1	-	-
	Sanctuary Drive		39.5	39.7	-	-
Caddies Creek Tributary 5	Old Windsor Road	TUFLOW	44.2	44.0	-	44.5
	NWRL (confluence with Caddies Creek)		44.1	44.0	-	-
Caddies Creek Tributary 4	Windsor Road	TUFLOW	44.6	44.9	-	44.6
	NWRL		43.8	43.6	-	-

Catchment	Location	Model Software	This Study	SKM, 2009 Note 1	JWP, 2010 Note 3	Maunsell, 2005 Note 3
	Confluence with Caddies Creek		41.1	40.5	-	-
Caddies Creek Tributary 3	Windsor Road	HEC-RAS	49.2	-	-	49.2
	NWRL		46.3	45.8	-	46.2
Second Ponds Creek	Schofields Road	HEC-RAS	48.3	47.1 Note 4	47.5 Note 4	-
	NWRL		45.7	46.0	46.1 Note 5	-
First Ponds Creek	NWRL	HEC-RAS	41.4	No comparison possible		

Notes:

1. Rouse Hill Integrated Stormwater Strategy Review – Hydrology, Hydraulics and Water Quality Review (Final Report), SKM, 10 July 2009
2. Area 20 Precinct, Rouse Hill Water Cycle Management Strategy Report Incorporating Water Sensitive Urban Design Techniques, J. Wyndham Prince, October 2010
3. North West Transitway Section 5: Drainage Southern End Final Design Revision 03 (28 September 2005)/ Drainage Northern End Final Design Revision 03 (13 December 2005), Maunsell
4. Makes some allowance for the proposed upgrade of Schofields Road, no date available on existing flood levels
5. Makes some allowance for the proposed NWRL, no date available on existing levels

B.3.2 Climate Change

As discussed in the Section 4.4, sensitivity analyses were carried out for increases in rainfall intensities of 10, 20 and 30%. As expected, the general trend is that peak flood levels tend to increase with corresponding increases in rainfall. However, the degree to which the different streams are affected differs considerably, and is most probably influenced by local hydraulic controls such as culverts, bridges and basins.

Table B.7. Potential Climate Change Relative Impacts (m) for 100 year ARI Peak Flood Levels

Location	Scenario (Increase in Rainfall Intensities)		
	+10%	+20%	+30%
Devlins Creek Tributary	+0.14-0.17m	+0.20-0.30m	+0.30-0.50m
Cattai Creek	+0.10-0.20m	+0.30-0.40m	+0.30-0.40m
Elizabeth Macarthur Creek	+0.02-0.15m	+0.07-0.30m	+0.10-0.45m
Caddies Creek	+0.05-0.30m	+0.10-0.50m	+0.15-0.70m
Caddies Creek Tributary 5	+0.03-0.05m	+0.06-0.10m	+0.08-0.15m
Caddies Creek Tributary 4	+0.05-0.06m	+0.10-0.11m	+0.15-0.16m
Caddies Creek Tributary 3	+0.03-0.06m	+0.04-0.11m	+0.07-0.17m
Second Ponds Creek	+0.05-0.08m	+0.11-0.16m	+0.15-0.28m
First Ponds Creek	+0.01-0.06m	+0.02-0.13m	+0.04-0.17m

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Appendix C

Appendix C - Preliminary Sediment Basin Sizing

Appendix C Preliminary Sediment Basin Sizing

Preliminary sediment basin sizings are provided in Table C1, based conservatively on disturbance of the entire construction sites. The construction of the stations, precincts and services facilities is tied in with the construction of the major civil works, as discussed in the EIS 1. The location of the construction sites for the EIS 2 works would be consistent with the sites required for the major civil construction works as described in EIS 1. The sizings for the preliminary sediment basins have therefore taken into consideration the construction of both the major civil works, as well as stations and precincts.

The volume required could be offset by the use of alternative measures such as bunded swales, staging of works and stabilisation of areas. On this basis, the estimated volumes provide an indication of the upper bound size of total basin volume required at each location.

Calculations were carried out in accordance with Section 6.3 of the Blue Book and are summarised in Tables C2 to C4.

Table C1 Preliminary Sediment Basin Sizing

Construction Site	Site Area (hectares)	Preliminary Sediment Basin Volume (m^3)
1. Epping Services Facility	0.34	130
3. Cheltenham Services Facility	1.2	450
4. Cherrybrook Station	7.5	2810
5. Castle Hill Station	1.8	680
6. Showground Station	6.5	2900
7. Norwest Station	2.1	790
8. Bella Vista Station	6.3	2360
9. Balmoral Road	19	7110
10. Memorial Avenue	12	2940
11. Kellyville Station	10	2450
12. Windsor Road/Old Windsor Road	5.0	1230
13. Old Windsor Road/Whitehart Drive	9.7	2380
14. Rouse Hill Station	1.8	440
15. Windsor Road Viaduct	6.1	1500
16.Windsor Road Viaduct to Cudgegong Road	8.9	2180
17. Cudgegong Road to Tallawong Stabling Facility	59	14450

Note: Site 2 was assessed as part of EIS 1 but does not form part of EIS scope of works.

Table C2 Preliminary Sediment Basin Calculations (Sites 1 to 6)

Site area	Site						Remarks
	1		3	4	5	6	
Total catchment area (ha)	0.34		1.2	7.5	1.8	6.5	
Disturbed catchment area (ha)	0.34		1.2	7.5	1.8	6.5	

Soil analysis

% sand (faction 0.02 to 2.00 mm							Soil texture should be assessed through mechanical dispersion only. Dispersing agents (e.g. Calgon) should not be used
% silt (fraction 0.002 to 0.02 mm)							
% clay (fraction finer than 0.002 mm)							
Dispersion percentage							E.g. enter 10 for dispersion of 10%
% of whole soil dispersible							See Section 6.3.3(e)
Soil Texture Group							See Section 6.3.3(c), (d) and (e)

Rainfall data

Design rainfall depth (days)	5		5	5	5	5	See Sections 6.3.4 (d) and (e)
Design rainfall depth (percentile)	85		85	85	85	85	See Sections 6.3.4 (f) and (g)
x-day, y-percentile rainfall event	43		43	43	43	43	See Section 6.3.4 (h)
Rainfall intensity: 2-year, 6-hour storm	12.1		12.1	12.1	12.1	12.1	See IFD chart for the site

RUSLE Factors

Rainfall erosivity (*R*-factor)	3160		3160	3160	3160	3160	Automatic calculation from above data
Soil erodibility (*K*-factor)	0.04		0.04	0.04	0.04	0.04	RUSLE data can be obtained from Appendices A, B and C
Slope length (m)	80		80	80	80	80	
Slope gradient (%)	10		10	10	10	10	
Length/gradient (*LS*-factor)	2.81		2.81	2.81	2.81	2.81	
Erosion control practice (*P*-factor)	1.3		1.3	1.3	1.3	1.3	
Ground cover (*C*-factor)	1		1	1	1	1	
Soil loss (t/ha/yr)	462		462	462	462	462	
Soil Loss Class	4		4	4	4	4	See Section 4.4.2(b)
Soil loss (m^3/ha/yr)	355		355	355	355	355	

Total Basin Volume

Site	C_v	$R_{x\text{-day, y-\%ile}}$	Total catchment area (ha)	Settling zone volume (m^3)	Sediment storage volume (m^3)	Total basin volume (m^3)
1	0.58	43	0.34	84.796	42	127.194
3	0.58	43	1.2	299.28	150	448.92
4	0.58	43	7.5	1870.5	935	2805.75
5	0.58	43	1.8	448.92	224	673.38
6	0.69	43	6.5	1928.55	964	2892.825

Table C3 Preliminary Sediment Basin Calculations (Sites 7 to 12)

Site area	Site						Remarks
	7	8	9	10	11	12	
Total catchment area (ha)	2.1	6.3	19	12	10	5	
Disturbed catchment area (ha)	2.1	6.3	19	12	10	5	
Soil analysis							
% sand (faction 0.02 to 2.00 mm							Soil texture should be assessed through mechanical dispersion only. Dispersing agents (e.g. Calgon) should not be used
% silt (fraction 0.002 to 0.02 mm)							
% clay (fraction finer than 0.002 mm)							
Dispersion percentage							E.g. enter 10 for dispersion of 10%
% of whole soil dispersible							See Section 6.3.3(e)
Soil Texture Group							See Section 6.3.3(c), (d) and (e)
Rainfall data							
Design rainfall depth (days)	5	5	5	5	5	5	See Sections 6.3.4 (d) and (e)
Design rainfall depth (percentile)	85	85	85	85	85	85	See Sections 6.3.4 (f) and (g)
x-day, y-percentile rainfall event	43	43	43	32	32	32	See Section 6.3.4 (h)
Rainfall intensity: 2-year, 6-hour storm	12.1	12.1	12.1	10.6	10.6	10.6	See IFD chart for the site
RUSLE Factors							
Rainfall erosivity (*R*-factor)	3160	3160	3160	2460	2460	2460	Automatic calculation from above data
Soil erodibility (*K*-factor)	0.04	0.04	0.04	0.038	0.038	0.038	RUSLE data can be obtained from Appendices A, B and C
Slope length (m)	80	80	80	80	80	80	
Slope gradient (%)	10	10	10	3	3	3	
Length/gradient (*LS*-factor)	2.81	2.81	2.81	0.65	0.65	0.65	
Erosion control practice (*P*-factor)	1.3	1.3	1.3	1.3	1.3	1.3	
Ground cover (*C*-factor)	1	1	1	1	1	1	
Soil loss (t/ha/yr)	462	462	462	79	79	79	
Soil Loss Class	4	4	4	1	1	1	See Section 4.4.2(b)
Soil loss (m^3/ha/yr)	355	355	355	61	61	61	

Total Basin Volume

Site	C_v	$R_{x\text{-day, }y\text{-\%ile}}$	Total catchment area (ha)	Settling zone volume (m^3)	Sediment storage volume (m^3)	Total basin volume (m^3)
7	0.58	43	2.1	523.74	262	785.61
8	0.58	43	6.3	1571.22	786	2356.83
9	0.58	43	19	4738.6	2369	7107.9
10	0.51	32	12	1958.4	979	2937.6
11	0.51	32	10	1632	816	2448
12	0.51	32	5	816	408	1224

Table C4 Preliminary Sediment Basin Calculations (Sites 13 to 17)

Site area	Site						Remarks
	13	14	15	16	17		
Total catchment area (ha)	9.7	1.8	6.1	8.9	59		
Disturbed catchment area (ha)	9.7	1.8	6.1	8.9	59		

Soil analysis

% sand (faction 0.02 to 2.00 mm							Soil texture should be assessed through mechanical dispersion only. Dispersing agents (e.g. Calgon) should not be used
% silt (fraction 0.002 to 0.02 mm)							
% clay (fraction finer than 0.002 mm)							
Dispersion percentage							E.g. enter 10 for dispersion of 10%
% of whole soil dispersible							See Section 6.3.3(e)
Soil Texture Group							See Section 6.3.3(c), (d) and (e)

Rainfall data

Design rainfall depth (days)	5	5	5	5	5		See Sections 6.3.4 (d) and (e)
Design rainfall depth (percentile)	85	85	85	85	85		See Sections 6.3.4 (f) and (g)
x-day, y-percentile rainfall event	32	32	32	32	32		See Section 6.3.4 (h)
Rainfall intensity: 2-year, 6-hour storm	10.6	10.6	10.6	10.6	10.6		See IFD chart for the site

RUSLE Factors

Rainfall erosivity (*R*-factor)	2460	2460	2460	2460	2460		Automatic calculation from above data
Soil erodibility (*K*-factor)	0.038	0.038	0.038	0.038	0.038		RUSLE data can be obtained from Appendices A, B and C
Slope length (m)	80	80	80	80	80		
Slope gradient (%)	3	3	3	3	3		
Length/gradient (*LS*-factor)	0.65	0.65	0.65	0.65	0.65		
Erosion control practice (*P*-factor)	1.3	1.3	1.3	1.3	1.3		
Ground cover (*C*-factor)	1	1	1	1	1		
Soil loss (t/ha/yr)	79	79	79	79	79		
Soil Loss Class	1	1	1	1	1		See Section 4.4.2(b)
Soil loss (m^3/ha/yr)	61	61	61	61	61		

Total Basin Volume

Site	C_v	$R_{x\text{-day, y-\%ile}}$	Total catchment area (ha)	Settling zone volume (m^3)	Sediment storage volume (m^3)	Total basin volume (m^3)
13	0.51	32	9.7	1583.04	792	2374.56
14	0.51	32	1.8	293.76	147	440.64
15	0.51	32	6.1	995.52	498	1493.28
16	0.51	32	8.9	1452.48	726	2178.72
17	0.51	32	59	9628.8	4814	14443.2