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.

Five 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)
- 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
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, which 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	-
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 – The Hills 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.

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 yard 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 yard 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
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 yard 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>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.

First Ponds Creek

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 Flood Levels (mAHD)

Catchment	Location	Model Software	This Study	SKM, 2009 [Note 1]	JWP, 2010 [Note 2]	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 2]	Maunsell, 2005 [Note 3]
	Confluence with Caddies Creek		41.1	40.5	-	-
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.

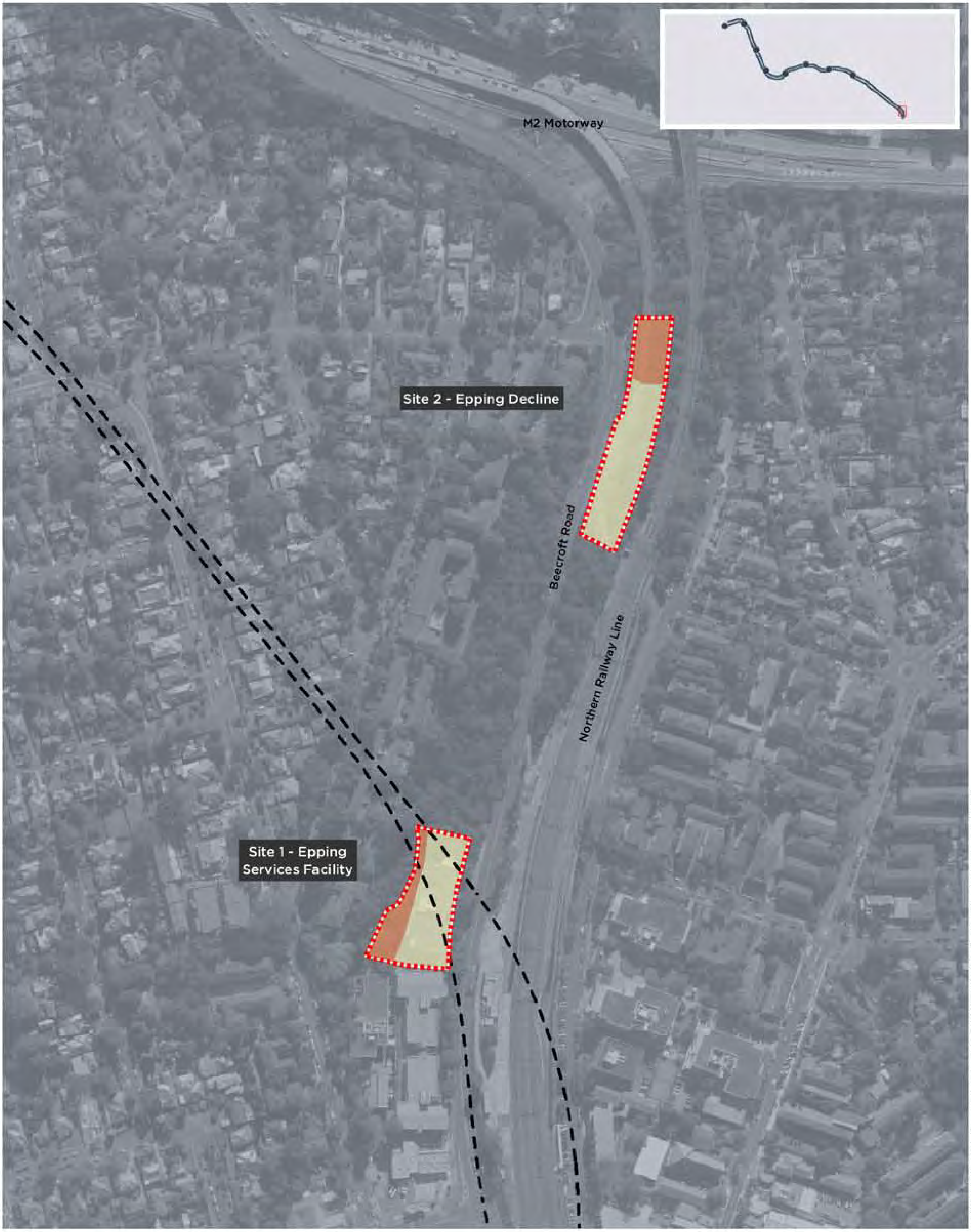
Table B.7. Potential Climate Change Relative Impacts (m) for 100 year ARI Peak Flood Levels

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
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

Appendix C

Appendix C – Preliminary Soil Risk Mapping

Appendix C Preliminary Soil Risk Mapping



NWRL alignment
- Below ground
- Above ground
- Above ground (bridge / viaduct)
- Construction site boundary

Soil risk
- High
- Moderate
- Low

0 25 50 100 m

NWRL alignment

- Below ground
- Above ground
- Above ground (bridge / viaduct)
- Construction site boundary

Soil risk

- High
- Moderate
- Low

0 25 50 100 m

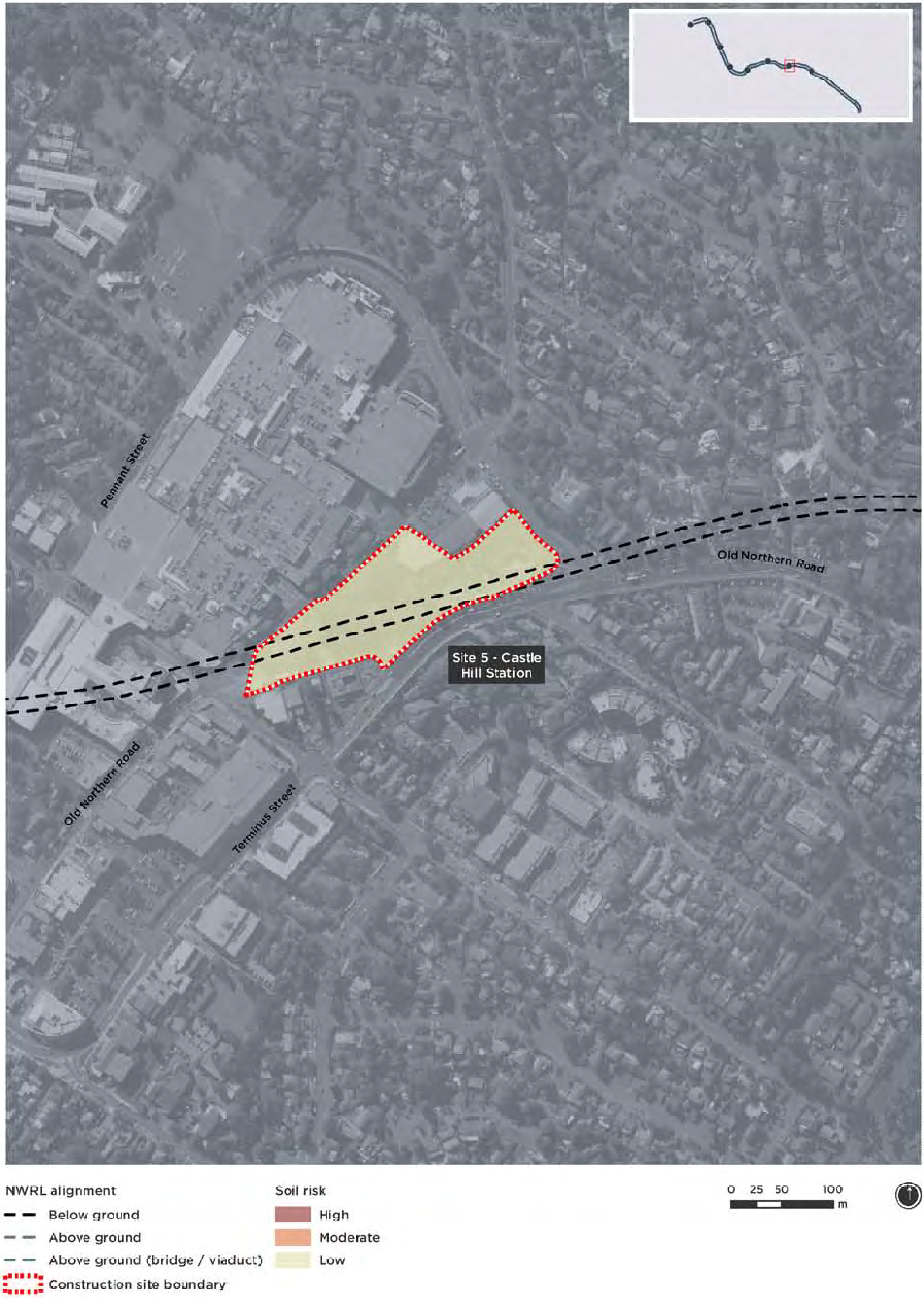
NWRL alignment

- Below ground
- Above ground
- Above ground (bridge / viaduct)
- Construction site boundary

Soil risk

- High
- Moderate
- Low

0 25 50 100 m



Pennant Street
Old Northern Road
Terminus Street
Old Northern Road
Site 5 - Castle Hill Station
NWRL alignment
Below ground
Above ground
Above ground (bridge / viaduct)
Construction site boundary
Soil risk
High
Moderate
Low
0 25 50 100
m

Showground Road
Site 6 - Hills Centre Station
NWRL alignment
Below ground
Above ground
Above ground (bridge / viaduct)
Construction site boundary
Soil risk
High
Moderate
Low
0 25 50 100
m

NWRL alignment

Below ground

Above ground

Above ground (bridge / viaduct)

Construction site boundary

Soil risk

High

Moderate

Low

0 25 50 100 m

NWRL alignment

- Below ground
- Above ground
- Above ground (bridge / viaduct)
- Construction site boundary

Soil risk

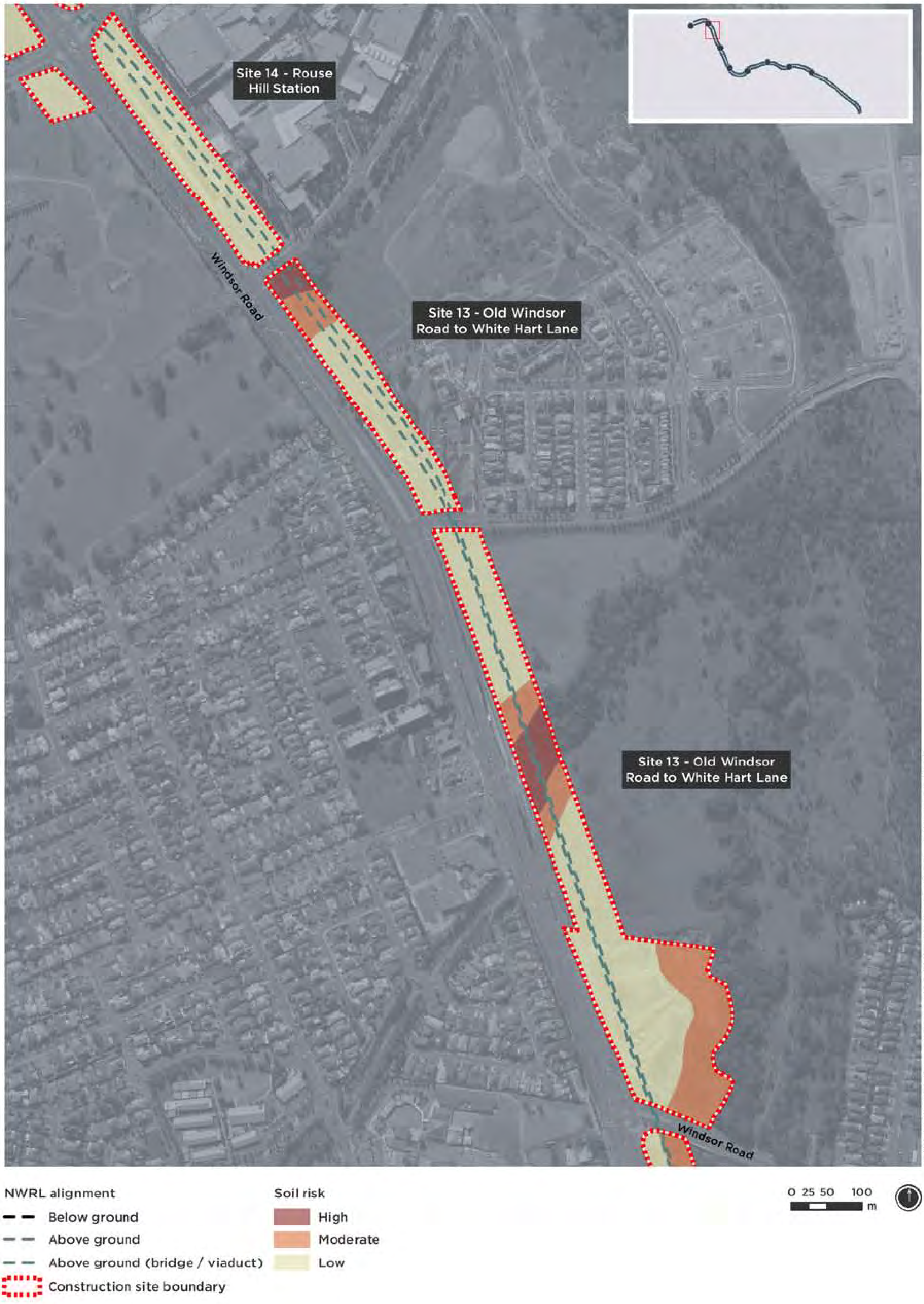
- High
- Moderate
- Low

0 25 50 100 m

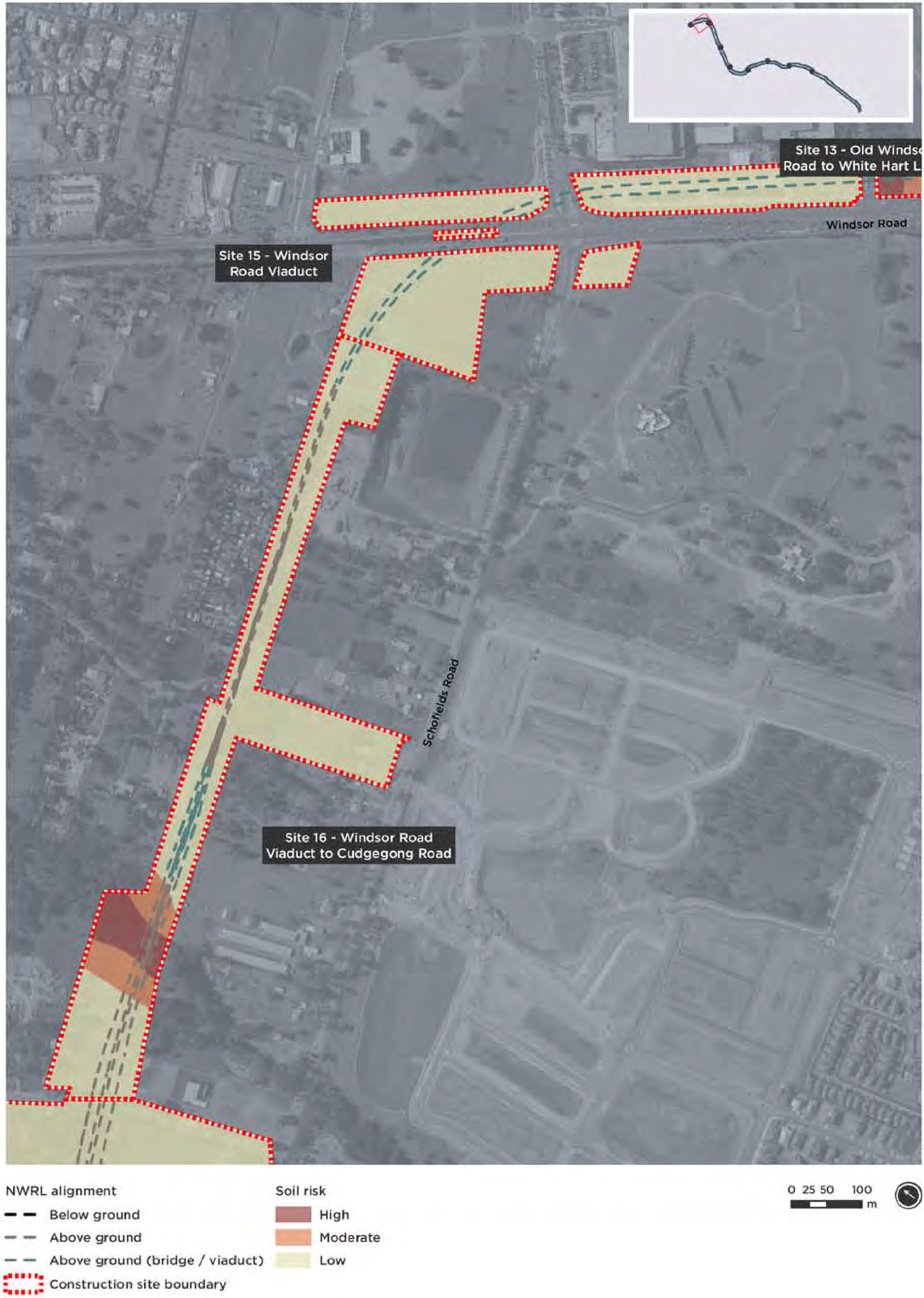
NWRL alignment
- Below ground
- Above ground
- Above ground (bridge / viaduct)
- Construction site boundary

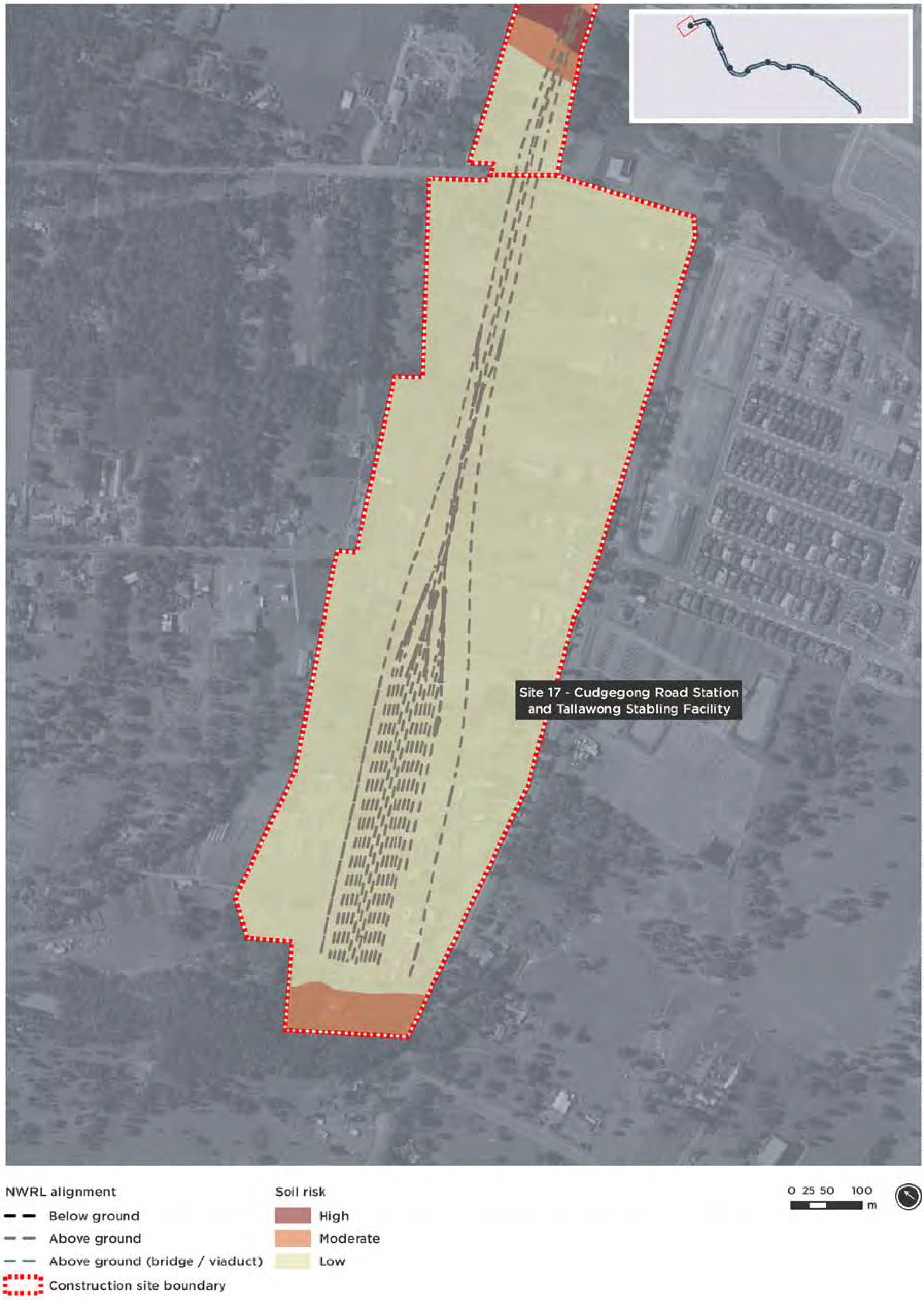
Soil risk
- High
- Moderate
- Low

0 25 50 100 m



Site 14 - Rouse Hill Station
Windsor Road
Site 13 - Old Windsor Road to White Hart Lane
Site 13 - Old Windsor Road to White Hart Lane
Windsor Road
NWRL alignment
Below ground
Above ground
Above ground (bridge / viaduct)
Construction site boundary
Soil risk
High
Moderate
Low
0 25 50 100
m


Site 13 - Old Winds
Road to White Hart L
Windsor Road
Site 15 - Windsor Road Viaduct
Schofields Road
Site 16 - Windsor Road Viaduct to Cudgegong Road
NWRL alignment
Below ground
Above ground
Above ground (bridge / viaduct)
Construction site boundary
Soil risk
High
Moderate
Low
0 25 50 100
m


Site 17 - Cudgegong Road Station and Tallawong Stabling Facility
NWRL alignment
Below ground
Above ground
Above ground (bridge / viaduct)
Construction site boundary
Soil risk
High
Moderate
Low
0 25 50 100
m

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Appendix D

Appendix D - Preliminary Sediment Basin Sizing

Appendix D Preliminary Sediment Basin Sizing

Preliminary sediment basin sizings are provided in Table D1, based conservatively on disturbance of the entire construction sites. 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 D2 to D4.

Table D1Preliminary Sediment Basin Sizing

Construction Site	Site Area (hectares)	Preliminary Sediment Basin Volume (m^3)
1. Epping Services Facility	0.34	130
2. Epping Decline	0.45	170
3. Cheltenham Services Facility	1.2	450
4. Cherrybrook Station	7.5	2810
5. Castle Hill Station	1.8	680
6. Hills Centre 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 Yard	59	14450

Table D2 Preliminary Sediment Basin Calculations (Sites 1 to 6)

Site area	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6	Remarks
Total catchment area (ha)	0.34	0.45	1.2	7.5	1.8	6.5	
Disturbed catchment area (ha)	0.34	0.45	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	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	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	12.1	See IFD chart for the site
RUSLE Factors							
Rainfall erosivity (*R*-factor)	3160	3160	3160	3160	3160	3160	Automatic calculation from above data
Soil erodibility (*K*-factor)	0.04	0.04	0.04	0.04	0.04	0.04	RUSLE data can be obtained from Appendices A, B and D
Slope length (m)	80	80	80	80	80	80	
Slope gradient (%)	10	10	10	10	10	10	
Length/gradient (*LS*-factor)	2.81	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	1.3	
Ground cover (*C*-factor)	1	1	1	1	1	1	
Soil loss (t/ha/yr)	462	462	462	462	462	462	
Soil Loss Class	4	4	4	4	4	4	See Section 4.4.2(b)
Soil loss (m³/ha/yr)	355	355	355	355	355	355	

Total Basin Volume

Site	C_v	$R_{x\text{-day, y-\%ile}}$	Total catchment area (ha)	Settling zone volume (m³)	Sediment storage volume (m³)	Total basin volume (m³)
1	0.58	43	0.34	84.796	42	127.194
2	0.58	43	0.45	112.23	56	168.345
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 D3 Preliminary Sediment Basin Calculations (Sites 7 to 12)

Site area	Site 7	8	9	10	11	12	Remarks
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 D
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	$R_{x\text{-day, y-\%ile}}$	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 D4 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 D
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	$R_{x\text{-day}, y\text{-\%ile}}$	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