Appendix D

Environmental Referral, North West infrastructure Multi User Iron Ore Export (Landside) Facility

Surface Water Report

North West Infrastructure Proposed Multi-User Iron Ore Export Facility

SURFACE WATER IMPACT ASSESSMENT COMPONENT OF ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT

- QE09828-EV-RP-0001-RevisionC
- 8 June 2011

Proposed Multi-User Iron Ore Export Facility

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- QE09828-EV-RP-0001-RevisionC
- 8 June 2011

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Executive Summary

Background

North West Infrastructure (NWI) has proposed a development of a multi-user iron ore export facility for Port Headland in Western Australia. The *Landside* project as described within the draft definitive feasibility study (DFS) comprises a wharf, overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the northern portion (approximately 5km) of the western rail spur. The Landside project ends at the southern boundary of the land proposed to be vested in the PHPA, and mirrors a similar infrastructure development by the Roy Hill Iron Ore (RHIO) recently approved by the Environmental Protection Authority (EPA 2011).

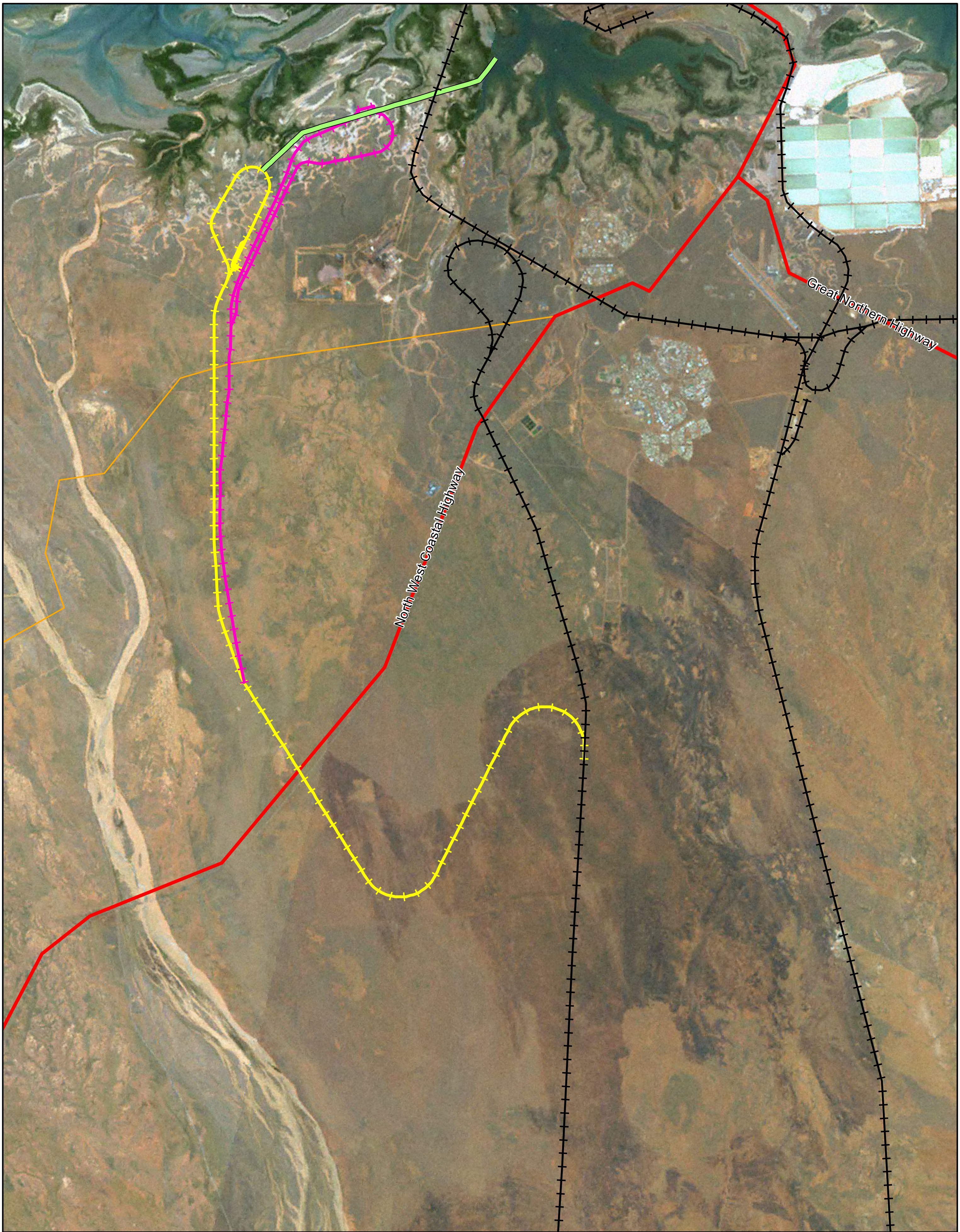
The *Rail* project involves a rail connection between the western rail spur and conections to any combination of rail lines operated or proposed by and the Fortescue Metals Group (FMG), BHP-Billeton Iron Ore (BHP-BIO) and RHIO rail lines. The alignment of existing infrastructure and the proposed NWI facilities are shown in Figure 1-1, with the FMG connection identified as a reasonable worst case and adopted as the base case for modelling in this report.

The proposed NWI rail loop, conveyor and port infrastructure is located immediately to the west of a similar railway loop, railway line, iron ore stockpile, conveyor and wharf facility that has been proposed by RHIO (Roy Hill Infrastructure Pty Ltd, 2010), as shown in Figure 1-1.

Floods and storm surges are known to occur in the vicinity of the project site. Such events may result in inundation of some of the proposed infrastructure and modifying the direction, depth and velocity of flows in areas around the project site. As shown in Figure 1-1, the proposed railway connecting line from the FMG line crosses the drainage line of South West Creek. The NWI and Roy Hill lines split after crossing South West Creek and travel parallel to one another and they also run parallel to the major drainage lines of South West Creek, South Creek and the Turner River.

The Project Area is located within the catchments of South and South West Creeks on the Pilbara Coast near Port Hedland. The Project area is in the Western part of this area and part of the indicated NWI railway alignment runs along the divide between the catchments of South West Creek and the Turner River.

South and South West Creeks drain into Port Hedland Harbour, which then drain into the Indian Ocean. The catchments of South and South West Creeks are sufficiently flat (particularly in the northern part of each catchment) that during periods of flood or high tides and storm surge the flows from the two creeks combine into a single drainage system. During flood periods flows from the catchment of South Creek can cross over and flow into South West Creek and vice versa. The combined catchment area of South and South West Creeks is 557 km^2.



Legend
Existing Railway
Proposed Railway and Loop
NWI
Roy Hill
Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 1-1 Locaility Plan

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The Turner River is a much larger catchment than the combined catchments of South and South West Creeks, having a total catchment area of 3556 km^2. The alignment of the proposed Project is such that no surface water flows are expected from the Project into the Turner River and that flows in the Turner River would not be modified by the Proposed Project.

Method

A combined hydrological and hydraulic model was used to investigate flooding under existing conditions and to assess impacts of the proposed development and also the combined impact of the NWI and Roy Hill developments. A hydrological model was established for the combined catchments of South and South West Creeks in the RORB rainfall runoff modelling program. Peak flows estimated from the RORB model were validated against flood peaks assessed from a regional frequency analysis of flood peaks recorded from other catchments of similar size in the Pilbara. A model of the area containing the Project site was created using a MIKE FLOOD dynamically coupled one-dimensional and two-dimensional hydraulic model.

Some parts of the Project site footprint would be expected to be inundated by storm surge and high tide, rainfall derived runoff in the catchments of South and South West Creeks or by a combination of storm surge and rainfall derived runoff. A Tropical Cyclone approaching or crossing the coast in the vicinity of Port Hedland is likely to lead to a combined storm surge and rainfall runoff flooding event in this area. The scenarios modelled to estimate peak flood levels were as shown in Table 1 Combined catchment runoff and storm surge events modelled.

- **Table 1 Combined catchment runoff and storm surge events modelled**

Nominal Overall AEP of Event	AEP of Creek Runoff Flood Event	Tidal and Storm Surge Boundary Condition
1 in 10 AEP	1 in 10 AEP	Mean High Water Spring Tide Level (constant head boundary)
1 in 50 AEP	1 in 50 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 50 AEP storm surge event, sinusoidally varying temporal pattern
1 in 100 AEP	1 in 100 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 100 AEP storm surge event, sinusoidally varying temporal pattern

The developed state was modelled by introducing the proposed NWI railway loop, conveyor alignment (embankment and trestle) and adopted railway spur line into the model. The developed state model also includes the proposed impact of the Roy Hill loop, conveyor and adopted railway spur line, so that the combined effects of both developments could be assessed.

Assessment of Flood Impacts

Even under existing conditions, combined storm surge and flood events with an AEP of 1 in 50 and 1 in 100 cause flooding of the floodplains of South and South West Creeks, although the towns of Wedgefield and South Hedland are largely above the 1 in 100 AEP flood level. There is some flooding along minor drainage lines within South Hedland and around Wedgefield in the 1 in 100 AEP event, but it is not anticipated that there would be any over-floor flooding of buildings within either town under existing conditions.

The modelling simulations demonstrated that the largest impact upon runoff is the impedance of surface water flow. If no mitigation measures were put in place, the NWI and RHIO railway spur lines, rail loops/ stockpiles and conveyors would constrain the movement of surface water flows.

The modelled NWI spur line runs approximately parallel to the RHIO alignment and crosses South West Creek. If unmitigated, rail embankments would cause flows from South West Creek to back up and pond along the upstream side of the alignment. The mitigation measures to avoid this are to provide adequate culvert capacity through the South West Creek crossing for the railway spur lines and to place contour embankments along the upstream side of the railway spur line and to the North of the South West Creek culvert crossing to force South West Creek flows underneath the railway at the crossing location.

If the conveyor were built entirely along an embankment, with no waterway openings, this would impede both flood runoff from South and South West Creeks flowing out to the ocean and storm surge penetrating inland from the ocean. Mitigation measures to sustain surface water flows at acceptable levels is to include culverts where embankments are to be used and elevate the conveyor structure on pylons to allow for flood and storm surge flows to pass underneath in critical flow areas.

The model was run for the 1 in 10, 1 in 50 and 1 in 100 AEP flood events including the proposed NWI and Roy Hill developments, with the proposed mitigation measures of the culverts where the adopted railway spur line crosses South West Creek, the training embankment extending upstream of the South West Creek culvert crossing for 1.6 km in a south easterly direction and with the conveyors for both the NWI and Roy Hill developments supported on pylons north east of the RHIO stockyard.

The proposed railway loops for NWI and Roy Hill impede the passage of storm surge for the 1 in 100 AEP storm surge events, such that there are widespread areas of reductions in peak storm surge levels to the south and east of the two proposed rail loops (as shown in Figure 4-14). The only areas where flood levels for the 1in 100 AEP event are projected to increase with the proposed NWI and Roy Hill developments are from the proposed culvert of the adopted railway spur line across South West Creek, for a distance of approximately 7 km downstream. No existing infrastructure would be affected by the increase in flood levels for the 1 in 100AEP event that would result from the proposed developments. Figure 4-13 shows that with the proposed NWI and Roy Hill developments, the towns of Wedgefield and South Hedland remain above the 1 in 100 AEP flood event.

Figure 4-15 shows that flow velocities for the 1 in 100 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek. For the 1 in 100 event with the proposed developments, the peak velocities reach 2.4 m/s near to the eastern end of the proposed Roy Hill railway loop. Roy Hill propose to install a revetment wall in this location to withstand velocities and flows from flooding and storm surge events. Peak velocities in the 1 in 100 AEP event near to the western edge of the existing FMG railway loop reach 1.2 m/s, which is the same as velocities at this location for the 1 in 100 event under existing conditions. Velocities in the 1 in 100 AEP event under with the proposed developments would exceed 1 m/s for a distance of approximately 1 km downstream of the proposed culvert crossing of South West Creek, with a maximum velocity of 2.0 m/s immediately downstream of the culvert. Scour protection may be required, particularly in the vicinity of the waterway openings in the conveyor and the culvert through the adopted railway spur line, to control erosion during flood and storm surge events.

Changes to Rainfall Runoff Rates

Installation of the a wharf, overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the northern portion (approximately 5km) of the western rail spur will cause increases in runoff rates from those that would be observed under the natural catchment conditions, due to increases in the impervious area. This represents 0.03% of the total combined catchment area of South and South West Creeks. This change in the volume of runoff produced is insignificant when compared with the overall total volume of runoff generated from the existing catchments of South and South West Creeks during flood events.

During minor rainfall events there will be additional runoff generated from these impervious areas that would otherwise runoff into areas surrounding the project site. Stormwater management practices, such as detention of runoff from impervious areas to reduce flow velocities and the prospects of scour, may be required to manage these local increases in runoff rates.

Impacts on Surface Water Quality

Rainfall on iron ore stockpiles and impervious surfaces around the rail loop and conveyor will cause runoff that may contain sediment and low levels of other contaminants. Silt trap and sedimentation basin facilities will be developed to trap sediments washed off the stockpile areas during rainfall events to prevent these materials flowing unmitigated into the receiving environment.

Modelling results indicate the velocities of flows in the floodplain around the site may change as a result of the development, although due to the relatively low level of encroachment of the project site into the floodplain these changes in velocity are expected to be small. Suspended sediment loads of downstream water may be increased if alteration of flow regimes results in scour velocities created downstream of the development. Areas where high flow velocities may be expected should be identified and appropriate measures taken to slow flow and contain sediments on site.

Management Measures

During rainfall events, some additional stormwater runoff will be generated from new impervious areas of the stockpiles, road train ring road, the twin cell rotary car dumper and the ancillary infrastructure buildings. On-site stormwater management measures may need to be implemented to detain the runoff produced from these areas and to minimise scour caused by direct runoff from these areas.

The stockpiles, car dumper and conveyor loader will be located above the 1 in 100 AEP flood and storm surge level, to minimise the probability of this infrastructure being inundated. In the event of a large flood or storm surge event that floods the rotary car dumper and the facilities for detaining water released during unloading of the rail cars, some sediment and other contaminants may be released into the environment. During a large flood event (rarer than 1 in 100 AEP), there would be large volumes of water inundating the flood plains of South and South West Creek so that there would be considerable dilution of the potentially contaminated water. Volumes of flood water would be sufficiently large that there would not be expected to be appreciable increases in the concentrations of contaminants.

Culverts will be designed to allow flood flows to pass through the at the South West Creek crossing during the 1 in 100 AEP event without causing inundation of the railway line. A regular programme of inspection and maintenance of the culverts would be implemented so that they continue to perform their function in passing flows during flood events. Waterway openings, in the form of bridges or culverts, will be designed in the conveyor embankment to pass the 1 in 100 AEP flood and storm surge event.

Baseline and continuous monitoring of sediment and other pollutants during construction and operation of the facility will be conducted to detect any changes in water quality due to these activities.

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1 Introduction

1.1 Background

North West Infrastructure (NWI) has proposed a development of a multi-user iron ore export facility for Port Headland in Western Australia. The proposed *Landside* project includes a two berth wharf, overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the northern portion (approximately 5km) of the western rail spur. The *Rail* project involves a rail connection between the western rail spur and conections to any combination of rail lines operated or proposed by FMG, BHP-BIO and RHIO rail lines as indicated in Figure 1-1.

Coffey Environments are compiling the Environmental and Social Impact Assessment (ESIA) referral document for the project. SKM were engaged by Coffey Environments on behalf of NWI to undertake the surface water hydrology input to the ESIA.

The proposed NWI rail loop, conveyor and port infrastructure is located immediately to the west of a similar railway loop, railway line, iron ore stockpile, conveyor and wharf facility previously referrd to the Environmental Portection Authority (EPA) (Roy Hill Infrastructure Pty Ltd, 2010) and subsequently approved in Bulletin 1380 (EPA 2011), as shown in Figure 1-1.

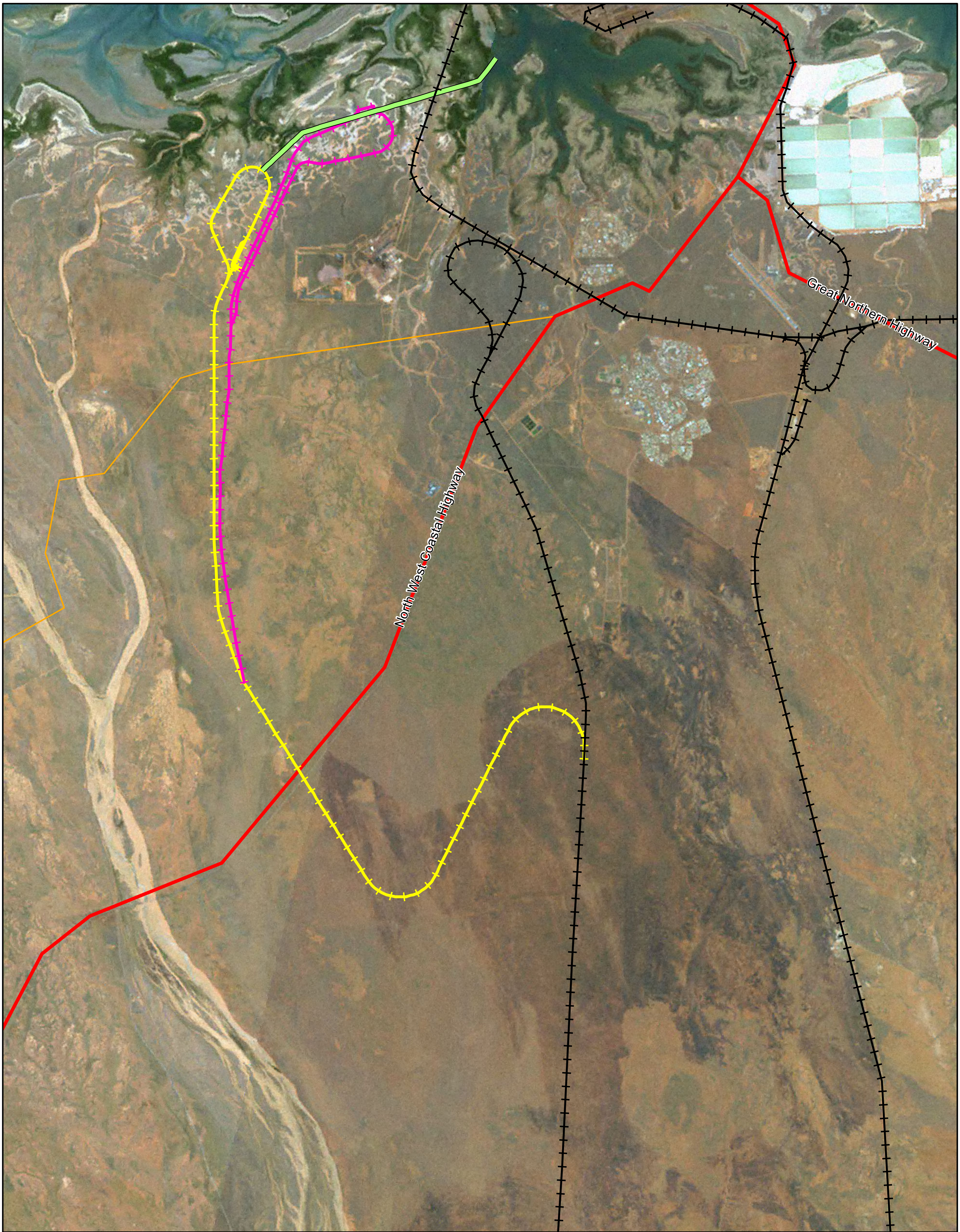
1.2 Scope of This Report

This report contains an assessment of the potential surface water impacts of the project and a discussion of mitigation measures to control those potential impacts. The terrain in the vicinity of the project site generally slopes from south to north, transitioning from broad, gently sloping plains to a flat intertidal zone to the north of the proposed NWI and Roy Hill railway loops.

Floods and storm surges are known to occur in the vicinity of the project site. Such events may result in inundation of some of the proposed infrastructure and modifying the direction, depth and velocity of flows in areas around the project site. As shown in Figure 1-1, the adopted railway connection from the FMG line crosses the drainage line of South West Creek. The NWI and Roy Hill lines split after crossing South West Creek and travel parallel to one another and in so doing also run parallel to the major drainage lines of South West Creek, South Creek and the Turner River.

This report uses a combined hydrological and hydraulic model to investigate flooding under existing conditions and to assess impacts of the proposed development and also the combined impact of the NWI and Roy Hill developments.

The proposed rail loop and conveyor are located on land with low elevation, often in the order of 1 to 2 m AHD. As a result, the proposed development will be subject to the influences of tidal cycles and storm tide effects. In addition to these, freshwater flooding from rainfall events do pass through the site, however generally these catchments are small and it is anticipated that the measures required to accommodate storm tide flows will comfortably accommodate the freshwater flows of equivalent recurrence interval. The adopted rail line from the rail loop across to the FMG railway line will also cross South West Creek, modifying flood flows from the catchment of South West Creek. The proposed Roy Hill Iron Ore infrastructure will have a combined influence on flooding in the area and the combined effects of the two proposed projects are assessed in this report.



Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

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North West Infrastructure

Figure 1-1 Locality Plan

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1.3 Reliance Statement

The sole purpose of this report and the associated services performed by Sinclair Knight Merz Pty Ltd (SKM) is to assess the impacts on surface water flows and water quality for the proposed overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the western rail spur for the Multi-User Iron Ore Export Facility in Port Hedland in accordance with the scope of services set out in the contract between SKM and Coffey Enviornments. That scope of services, as described in this report, was developed Coffey Environments

In preparing this report, SKM has relied upon, and presumed accurate, certain information (or absence thereof) provided by the Client and other sources. Except as otherwise stated in the report, SKM has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

SKM derived the data in this report from a variety of sources. The sources are identified at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. SKM has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose of the project and by reference to applicable standards, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report.

This report should be read in full and no excerpts are to be taken as representative of the findings. No responsibility is accepted by SKM for use of any part of this report in any other context. Whilst high resolution and accurate LIDAR derived terrain data was used for hydraulic modelling across much of the study area, one part of the area modelled distant from the proposed rail loop was modelled using lower resolution terrain data derived from Landgate 10 metre contour interval data. Whilst this has insignificant impact on the depths and velocities in the vicinity of the proposed Project, depths and velocities derived in and close to the area of lower resolution terrain data will have a lower level of accuracy.

This report has been prepared on behalf of, and for the exclusive use of, Coffey Environments and NWI, and is subject to, and issued in connection with, the provisions of the agreement between SKM and Coffey Environments and NWI. SKM accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party.

2 Source Data Analysis

2.1 Climate Data

2.1.1 Rainfall Data

Monthly rainfall statistics for Port Hedland Airport were obtained from the Bureau of Meteorology's Climate Data On-line service. Table 2 shows the monthly rainfall statistics for this site for the entire period of climate record (1942-2011). Port Hedland is a generally arid area, with mean annual rainfall of 310 mm/year. The statistics show that there is a pronounced wet season, running from mid-December through to about June, with mean monthly rainfall totals exceeding 10 mm/month. The maximum monthly rainfall totals and maximum daily rainfall totals are observed in the months between December and April, which aligns with the period when Tropical Cyclones typically occur within the region. The period from July through November is consistently dry, with low rainfall totals, although thunderstorms can occur at any time of the year. Year to year variability and the general aridity of the area can result in entire months with zero recorded rainfall, even during the period of the year that is typically classified as the wet season.

- **Table 2 Monthly rainfall statistics for Port Hedland airport (derived from entire period of record 1942-2011)**

Statistic	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Mean monthly rainfall (mm)	59.4	93.2	48.7	23.0	27.2	21.2	11.1	5.0	1.3	0.9	2.5	18.4	310.9
Median monthly rainfall (mm)	21.4	74.6	15.7	2.3	8.3	6.8	2.7	0.6	0.4	0.2	0.0	0.5	307.1
Maximum monthly rainfall (mm)	453.5	360.0	427.2	352.1	169.9	128.6	80.5	58.6	27.4	8.2	66.8	219.0	626.8
Minimum monthly rainfall (mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	44.5
Highest recorded daily total in month (mm)	387.1	328.9	156.8	117.2	156.2	127.6	73.2	34.6	19.0	7.4	59.4	169.3	387.1
Mean number of days of rain	5.0	7.1	4.4	2.0	3.1	2.9	2.2	1.2	1.0	0.8	0.6	1.9	32.2

Design values adopted for flood modelling were from databases of design rainfall that are accepted industry standard for flood modelling, Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) and Cooperative Research Centre Focussed Rainfall Growth Estimation method for Western Australia (Durrant and Bowman, 2004).

2.1.2 Tidal and Storm Surge Data

The Indian Ocean in the Port Hedland area undergoes significant tidal variation. The Highest Astronomical Tide defines the tide level produced by the combined effects of the solar and lunar cycles only, with no allowance for storm surge conditions. Under these conditions, the peak water level in Port Hedland reaches 4.0 m AHD. The solar and lunar cycles are such that the Highest Astronomical Tide only occurs every 18.6 years.

Spring tides occur approximately every two weeks (at the time of full and new moons). The Mean of High Water Spring (MHWS) tides, estimated using the astronomical components only, is 2.83 m AHD at Port Hedland. For modelling the combined effects of relatively common flood events (for example the 1 in 10 annual exceedance probability (AEP)[1] event), using a downstream boundary condition of MHWS at 2.83 m AHD provides a reasonable estimate of the combined peak water levels of an event with an overall annual exceedance probability (AEP) of 1 in 10.

In addition to tidal effects, storm surges caused by Tropical Cyclones and other tropical storms will elevate water levels in the ocean above the tidally induced water level range. Global Environmental Modelling Systems (2000) includes a summary table of observed flood events in the Port Hedland region between 1917 and 2000. The peak storm surge level recorded in Port Hedland of 5.7 m in January 1939 caused inundation of the Hotel in Port Hedland (Jacka, 1994).

Global Environmental Modelling Systems (2000) used a hydrodynamic modelling approach to estimate storm surge levels in Port Hedland with AEP of 1 in 50 and 1 in 100. Global Environmental Modelling Systems (2000) identifies that the peak storm surge levels for at the Ocean in the vicinity of Port Hedland are 5.5 and 7.4 m AHD respectively.

2.2 Topography

The primary source of topographic data for the hydraulic model was a digital terrain model (DTM) derived using light detection and ranging (LIDAR), which covered the alignment of the proposed rail loop and connection to the FMG rail line. The LIDAR terrain data coverage has a horizontal resolution of 10 metres and a vertical accuracy (to one standard deviation) of +/- 100 mm.

Due to limitations in the extent of LIDAR data coverage for the DTM, levels for one part of the terrain were derived from lower resolution contour data from Landgate Western Australia. This dataset was derived from contour data with a contour interval of 10 metres and that was originally mapped on topographic maps with a scale of 1:50,000.

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[1] The probabilty that a given flood peak or rainfall total accumulated over a given duration will be exceeded in any one year. Bureau of Meteorology explain the relationship between the AEP and ARI at http://www.bom.gov.au/hydro/has/ari_aep.shtml .

For the purposes of hydraulic modelling the two data sets were merged into a single DTM. Using the reasonable assumption that the LIDAR data is considerably more accurate than the Landgate contour data, this process of merging required some smoothing of the Landgate derived data near the intersection with the LIDAR data to provide for a sufficiently smooth transition in the terrain. Smoothing of the terrain in this way minimised artefacts in the hydraulic modelling results that could have resulted from sharp transitions in the DTM near the boundaries of the two data sets.

Catchments and subcatchments for the hydrological models of South and South West Creeks were derived using a combination of the Landgate 10 metre contour data (for the upper catchment) and the combined DTM described above that was also used for the hydraulic modelling. This data resolution provides sufficient resolution to define the drainage for each subcatchments in the hydrological model to sufficient accuracy.

The DEM data finished at the coastline. The coastline was therefore defined as a water level elevation boundary in the hydraulic model with water levels forced according to tidal and storm surge levels in the ocean.

2.3 Vegetation, Landuse and Landform

Vegetation across the catchments of South and South West Creek mainly consists of scattered shrub and grasslands. There are mangrove communities along the coastline in and near the inter-tidal zone.

The Project site is located across several pastoral leases and has been subject to cattle grazing in the past. Aerial photography and satellite imagery of the project area, along with a field survey, was used to assess catchment roughness values that were adopted in the hydraulic modelling.

2.4 Adopted Railway Spur Line, Rail Loop and Conveyor Alignment

Figure 4-5 shows the adopted railway spur line, rail loop and conveyor alignment associated with the proposed project. The adopted railway spur line, railway loop and conveyor will all be built up to above the 1 in 100 AEP flood level. Iron ore stockpiles will be located within the lower area in the centre of the railway loop. Drainage from the centre of the railway loop will be designed so that rainfall occurring within the rail loop can drain out but flap gates will be installed to prevent storm surge, floods and tides running into the centre of the rail loop where the iron ore stockpiles will be located.

3 Flood Hydrology

3.1 Catchment Description

The Project Area is located within the catchments of South and South West Creeks on the Pilbara Coast near Port Hedland. The Project area is in the Western part of this area and part of the proposed NWI adopted railway alignment runs along the divide between the catchments of South West Creek and the Turner River.

South and South West Creeks drain into Port Hedland Harbour, which then drain into the Indian Ocean. The catchments of South and South West Creeks are sufficiently flat (particularly in the northern part of each catchment) that during periods of flood or high tides and storm surge the flows from the two creeks combine into a single drainage system. During flood periods flows from the catchment of South Creek can cross over and flow into South West Creek and vice versa. The combined catchment area of South and South West Creeks is 557 km^2.

The Turner River is a much larger catchment than the combined catchments of South and South West Creeks, having a total catchment area of 3556 km^2. The alignment of the proposed Project is such that no surface water flows are expected from the Project into the Turner River and that flows in the Turner River would not be modified by the Proposed Project.

3.2 Hydrological Analysis Method

The hydrology of the project site was analysed using the RORB rainfall runoff routing model (Laurenson and Mein, 1992). The combined catchment of South and South West Creeks was subdivided into eighteen separate subcatchments, with subcatchments divided using the combined DTM from the Landgate 10 metre contour data and the LIDAR derived DTM for the proposed Project Area.

The RORB model was run for design rainfall events with AEP of 1 in 10, 50 and 100 and for rainfall event durations of 6, 12 and 24 hours. Since the level of urbanisation is low across the catchment, the entire catchment was modelled as pervious.

3.3 Catchment Design Rainfall Estimates

Design rainfall estimates for the combined catchment of South and South West Creek were estimated using established databases of design rainfalls for use in Western Australia. Design rainfall for 24 hour duration and 1 in 50 and 1 in 100 AEP events for catchment were determined from CRC FORGE database for WA (Durrant and Bowman, 2004) because this provides the most up to date and appropriate estimates for events in this AEP range. Areal reduction factors for all durations were determined from CRC FORGE work for WA (Durrant and Bowman, 2004), using the summer “rest of WA” equation.

Rainfall intensities for the catchment are derived for 24 hour duration events with AEP of 1 in 50 and 1 in 100 using the equation

CRC FORGE (Durrant and Bowman, 2004) only provides areal reduction factor (ARF)[2] and point design rainfall intensities for rainfall events with durations of 24 hours and longer. Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) provides ARF and point design rainfall intensities for shorter durations and AEP more common than 1 in 50 AEP. The estimates from Australian Rainfall and Runoff (1998) for design rainfall intensities and ARF have been effectively superseded by Durrant and Bowman (2004). However, the ratios of both ARF and design point rainfall intensity provide a useful means of extrapolating the design rainfall intensities for shorter durations from the estimates for 24 hours. Design intensities for the catchment for shorter durations were therefore determined using the following equation:

Table 3 lists the design rainfall depths for the AEP and the durations that were considered in hydrological modelling for the catchments of South and South West Creeks.

- **Table 3 Adopted design rainfall depths for design events on combined catchment area of South and South West Creeks**

Event Duration (hours)	Design Rainfall Depth in mm for Event with AEP			
	1 in 10	1 in 20	1 in 50	1 in 100
6	89	114	151	175
12	124	161	215	251
24	143	186	249	292

Temporal patterns for the design rainfall were derived from the patterns in Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) for Zone 7.

The spatial pattern of rainfall during the design events was assumed to be uniform across the 557 km^2 combined catchment area of South and South West Creeks.

[2] Ratio between the design rainfall intensity for a given burst duration and annual exceedance probability for a particular catchment area to the design rainfall intensity for the same duration and annual exceedance probability at a point.

3.4 Verification of RORB Model to Regional Flood Frequency Analysis

There are no streamflow gauging stations within the catchments of South or South West Creeks. It was therefore not possible to perform direct calibration of the RORB model to observed flood events within the catchment.

The parameters of the RORB model were verified to flood peak quantiles estimated from a regional frequency analysis of peak flows recorded from five gauging stations in the Fortescue River and Portland Coast Basins. Sites with recorded flow data were selected from those available in those basins, on the basis of sites that had at least 20 years of streamflow record and catchment areas between 40 and 1200 km^2. These gauging stations are as listed in Table 4.

- **Table 4 Streamflow Gauging Stations Used in Regional Flood Frequency Analysis**

Site Number	Site Name	Catchment Area (km²)	Period of Available Data	Years of Peak Flow Record Used in Analysis
709007	Harding River at Marmurrina Pool U-South	49	Aug. 1974 – May 1999	24
709006	Tanberry Creek at Blue Dog Pool	128	Mar. 1974 – May 2001	27
708227	Portland River at Recorder Pool	553	Nov. 1966 – May 2001	35
709010	Turner River at Pincunah	885	Jan. 1985 – Still open	18
709001	Harding River U/S Cooya Pooya	1058	Dec. 1965 – Jan. 1985	20

Regional flood frequency analysis was used to fit a combined flood frequency curve to the data from the five sites. L-Moment analysis (Hosking and Wallis, 1997) was used to fit a Generalised Extreme Value (GEV) distribution to the annual maxima for each of the sites. The fitting procedure used weighted averages of the higher order L-Moments so that the regional frequency curve has a consistent shape to the pooled data from all five sites. A regression equation was fitted between catchment area and the mean annual flood (first L-Moment value) derived for each of the five regional sites. The regression equation with catchment area was then used to determine the mean annual flood for the South and South West Creeks catchment, with the shape and parameters of the GEV distribution derived from the regional flood frequency analysis to derive peak flows for design floods for the combined catchment area of South and South West Creek. Peak flows for design events from the regional frequency analysis are provided in Table 5.

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- **Table 5 Verification of flood peaks produced by the RORB model at the combined outlet of South and South West Creeks to flood peaks estimated from regional flood frequency analysis**

Event	Growth Factor from Regional Frequency Analysis*	Regional Frequency Analysis Peak Flow Estimate (m^3/s)	RORB Model Peak Flow Estimate (m^3/s)
Mean Annual Flood	1	299	Not modelled
1 in 10 AEP	2.28	681	662
1 in 50 AEP	4.85	1450	1420
1 in 100 AEP	6.44	1925	1876

* Growth factor = Ratio of peak for a given AEP derived from the regional frequency analysis to the Mean Annual Flood

The RORB model was run for design floods with AEP of 1 in 10, 50 and 100. The value of the catchment delay parameter, k_c, for the RORB model was derived from the regional prediction equation contained in Book V of Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) for the Northwest, Kimberley and Wheatbelt regions, providing a value of $k_c = 32$. In accordance with the regional equation in Book V of Australian Rainfall and Runoff, the catchment non-linearity parameter m was set to a value of 0.8. The initial and continuing loss parameter values were adjusted until a fit was obtained between the peak flows derived from the RORB model and the peak flows derived from the regional frequency analysis. This verification process resulted in adopting an initial loss of 37 mm and a continuing loss of 7 mm/h. Flood peaks estimated by the RORB model for the design events are within 3% or less of peak flows from the regional flood frequency analysis, as shown in Table 5. While the continuing loss parameter value in particular is high, it is consistent with the loss rate that would be expected from a catchment in the arid Pilbara region. For all AEP that were modelled using RORB, the critical duration was 12 hours.

3.5 Hydrographs for Design Floods

The RORB model was run with the adopted parameters produced from the verification to the regional flood frequency analysis. Design flood hydrographs were extracted for the event with the critical duration that matched the peak flows from the regional flood frequency analysis. Separate flood hydrographs were extracted for each of the eighteen individual subcatchments in the RORB model. These were then used as direct inputs to the MIKE FLOOD hydraulic model.

4 Flooding Characteristics of the Project Area

4.1 Overview of Approach

Hydraulic modelling was used to assess the drainage characteristics of the Project site during several different flood scenarios. A model of the area containing the Project site was created using a MIKE FLOOD dynamically coupled one-dimensional and two-dimensional hydraulic model.

Some parts of the Project site footprint would be expected to be inundated by storm surge and high tide, rainfall derived runoff in the catchments of South and South West Creeks or by a combination of storm surge and rainfall derived runoff. A Tropical Cyclone approaching or crossing the coast in the vicinity of Port Hedland is likely to lead to a combined storm surge and rainfall runoff flooding event in this area. The response time of the catchments of South and South West Creeks are sufficiently short (approximately 12 hours critical duration) that it is likely that the storm surge and tidal flooding will peak at around the same time as the peak of rainfall runoff generated flooding in South and South West Creeks. The flood modelling scenarios adopted therefore consider a coincident storm surge and creek flooding event. For a given AEP, this approach is likely to produce conservative estimates of peak flood levels because there is some likelihood that the peaks of storm surge and river flooding events will not coincide. The scenarios modelled to estimate peak flood levels were as shown in Table 6.

Table 6 Combined catchment runoff and storm surge events modelled

Nominal Overall AEP of Event	AEP of Creek Runoff Flood Event	Tidal and Storm Surge Boundary Condition
1 in 10 AEP	1 in 10 AEP	Mean High Water Spring Tide Level (constant head boundary)
1 in 50 AEP	1 in 50 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 50 AEP storm surge event, sinusoidally varying temporal pattern
1 in 100 AEP	1 in 100 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 100 AEP storm surge event, sinusoidally varying temporal pattern

The developed state was modelled by introducing the proposed NWI railway loop, conveyor embankment and adopted railway spur line into the model. The developed state model also includes the proposed impact of the Roy Hill loop, conveyor embankment and adopted railway spur line, so that the combined effects of both developments could be assessed. The terrain layer of the digital terrain model was modified by increasing the terrain height to well above the projected 1 in 100 AEP flood level, so that the railway line will not be inundated by such an event. Openings were then introduced into the model to represent culvert or bridge structures to pass flows through the adopted railway spur line and underneath the conveyor. Finally, the terrain was modified by introducing training embankments that will direct flows from South West Creek through the waterway at the creek crossing of the spur line instead of allowing those flows to continue along the upstream side of the adopted railway spur line.

4.2 Base Case Flood Modelling

4.2.1 Model Development

Hydraulic modelling was used to evaluate the flooding characteristics of the Project site in its existing state and assess the potential impact of flood flows on the proposed plant site and access road. SKM used MIKE FLOOD, a dynamically coupled MIKE 21 two-dimensional and MIKE 11 one-dimensional hydrodynamic model developed by DHI, to give an accurate simulation of the hydraulic behaviour of the Project site.

4.2.2 Terrain data Inputs

MIKE 21 is a two-dimensional hydraulic model, which represents the broader topography of the Project area in the form of a model grid. As discussed in Section 2.2, terrain data for the hydraulic modelling domain was derived mainly from LIDAR data captured for the Project, with additional data for missing regions on the outskirts of the project area derived from Landgate 10 metre contours. The horizontal grid resolution of the DTM for the hydraulic model was 10 metres.

4.2.3 Runoff Hydrograph Inputs

Rainfall runoff hydrographs from South and South West Creeks upstream of the Project site and from the Local sub-catchments were input into the model grid in a number of different locations to represent the distribution of rainfall runoff over the project site.

4.2.4 Tidal and Storm Surge Boundary Condition

The Indian Ocean in the Port Hedland area undergoes significant tidal variation. The Highest Astronomical Tide defines the tide level produced by the combined effects of the solar and lunar cycles only, with no allowance for storm surge conditions. Under these conditions, the peak water level in Port Hedland reaches 3.6 m AHD. The solar and lunar cycles are such that the Highest Astronomical Tide only occurs every 18.6 years.

Spring tides occur approximately every two weeks (at the time of full and new moons). The Mean of High Water Spring (MHWS) tides, estimated using the astronomical components only, is 2.83 m AHD at Port Hedland. For modelling the combined effects of relatively common flood events (for example the 1 in 10 AEP event), using a downstream boundary condition of MHWS at 2.83 m AHD provides a reasonable estimate of the combined peak water levels of an event with an overall AEP of 1 in 10.

In addition to tidal effects, storm surges caused by Tropical Cyclones and other tropical storms will elevate water levels in the ocean above the tidally induced water level range. Global Environmental Modelling Systems (2000) includes a summary table of observed flood events in the Port Hedland region between 1917 and 2000. The peak storm surge level recorded in Port Hedland of 5.7 m in January 1939 caused inundation of the Hotel in Port Hedland.

Global Environmental Modelling Systems (2000) used a hydrodynamic modelling approach to estimate storm surge levels in Port Hedland with AEP of 1 in 50 and 1 in 100. The results of the Global Environmental Modelling Systems (2000) hydrodynamic modelling of the storm surges were used to inform the boundary condition hydrographs at the Indian Ocean. Storm surge levels from Global Environmental Modelling Systems (2000) vary from east to west across the coastal boundary of the model domain. Sinusoidally varying storm surge and tidal boundary conditions were entered into the model with the following peak water levels for the 1 in 50 AEP and 1 in 100 AEP design flood and storm surge scenarios as shown in Table 7.

- **Table 7 Modelled Peak Storm Surge Level Hydrographs at Coastline of Hydraulic Model (from Global Environmental Modelling Systems, 2000)**

Model Scenario	Peak Storm Surge Level at Coastline (m AHD)		
	Western End of Model Domain	Middle of Model Domain (near proposed rail loop)	Eastern End of Model Domain (Port Hedland Harbour)
1 in 50 AEP Storm Surge	7.5	5.5	5.35
1 in 100 AEP Storm Surge	8.6	7.4	6.0

4.2.5 Roughness and Eddy Viscosity

Whilst there are local variations in vegetation cover across the study area, there is generally scattered shrub vegetation across most of the study area. It was therefore assumed that there would be consistent hydraulic roughness across the study area in the hydraulic model. A consistent Manning's n value of 0.032 was adopted across the entire study area. A consistent value of eddy viscosity of 2 m^2/s.

4.2.6 Representation of Railway Lines, Roads and Structures

There are a number of existing road and railway crossings of waterways within the study area. Nine of these waterway crossings were represented in the MIKE-FLOOD model using one dimensional hydraulic modelling structure elements. Dimensions for culverts and bridge water way openings for each of these structures were determined from field inspection undertaken of the study area in February 2011.

The impacts on flood flows of the proposed NWI conveyor and conveyor pylons from the rail loop to the wharf have not been explicitly represented in the hydraulic model. The conveyor will be designed to be above the 1 in 100 AEP storm surge level and will not impact upon flood flows. Pylons to support the conveyor will be sufficiently small in diameter and at sufficiently wide spacing to have minimal impact on overall flood levels upstream of the project site. The proposed Roy Hill conveyor will also be elevated and supported on pylons, with pylons for their conveyor spaced at intervals of approximately 50 metres (Worley Parsons, 2010). Flood levels in the local vicinity of the conveyor pylons (for both the NWI and Roy Hill conveyors) may be up to 50 mm higher than indicated by the overall flood modelling.

4.2.7 Assumptions

At the time of this assessment no recorded historic flood levels could be obtained for the Project site. This lack of data prevented the hydraulic model from being calibrated against recorded flood data. For the purposes of this assessment it is assumed that all rainfall runoff inputs and tidal data used in the modelling are accurate, and therefore the model is a sufficiently reliable representation of the hydraulic characteristics of the Project site and the floodplains of South and South West Creek.

The accuracy of results obtained from the hydraulic model is determined by the accuracy of the raw digital elevation data used. An assumption has been made during this assessment that the topography supplied for this assessment as well as the interpolation of that raw data into a topographic grid is an accurate representation of the existing topography of the Project site. This assumption was necessary as no ground truthing was able to be conducted at the Project site during this assessment.

The accuracy of model results is also dependent on the accuracy of runoff hydrographs input into the model and water output from the model at the downstream boundary. The has been no streamflow gauging in the catchments of South or South West Creeks, so parameter values for the hydrological model were verified to regional flood frequency analysis. This provides a degree of confidence in the runoff input into the model. The fixed sea level boundary used in all simulations has not been verified with observations or historic records of actual tidal ingress into the Project site. The assumption that the tide accesses the Project site to the extent estimated may be a conservative over estimate of tidal heights at the site.

The accuracy of flood levels and depths obtained from modelling results for this assessment is limited by these assumptions, which present the possibility of a compound error. This error is estimated to be relatively small due to the use of accurate topographic data (accuracy within 0.1 m), and the use of well established design event rainfall runoff estimation methods.

In terms of comparing the base case and developed case modelling results, the errors are the same, or in other words like is being compared with like. This results in a high level of accuracy when evaluating the changes to flood waters which may potentially result from the proposed development.

4.2.8 Base Case Modelling Results

The purpose of the base case assessment was to evaluate the flooding characteristics of the Project site in its existing state, prior to the commencement of the proposed Project. Three different rainfall design events and storm surge floods were simulated, with nominal AEP of 1 in 10, 50 and 100. The adoption of high downstream tailwater conditions in conjunction with flood inflows produces conservatively high inundation extents for each of the simulated events.

In the 1 in 10 AEP event under existing conditions, Figure 4-1 shows that the combined high tide and storm surge floods the intertidal zone to the north of the proposed NWI and Roy Hill railway loops. Flooding also occurs across a broad area of the floodplain of South West Creek, to the east of the existing FMG railway line, and a broad area of the floodplain of South Creek, between the FMG and BHP Billiton Iron Ore railway lines. Depths of flooding across the floodplains in the 1in 10 AEP event are less than 1 metre although there would be depths of flow of up to 3 metres in the centre of the channel of South West Creek. The towns of Wedgefield and South Hedland are above the 1 in 10 AEP flood level under existing conditions. Figure 4-2 shows that modelled velocities of flow for the 1 in 10 AEP event across the most of the floodplain areas of South and South West Creeks are less than 1 m/s.

Figure 4-3 shows that the 1 in 50 AEP storm surge under existing conditions would flood the entire area where NWI and Roy Hill are proposing to locate their railway loops and iron ore stockpiles. In the 1 in 50 AEP event under existing conditions, broad areas of the floodplains of South West and South Creeks would be inundated, although the depth of inundation is typically less than 1 metre, except in the middle of the drainage channels for the two creeks. The towns of Wedgefield and South Hedland are above the 1 in 50 AEP flood level under existing conditions. Figure 4-4 shows that modelled velocities of flow for the 1 in 50 AEP event across the most of the floodplain areas of South and South West Creeks are less than 1 m/s.

The 1 in 100 AEP storm surge for existing conditions (shown in Figure 4-5) demonstrates a similar pattern to the 1 in 50 AEP event, with the entire area proposed for the NWI and Roy Hill railway loops and iron ore stockpiles being flooded. The 1 in 100 AEP event results in flooding of the floodplains of South and South West Creeks, although the towns of Wedgefield and South Hedland are largely above the 1 in 100 AEP flood level. There is some flooding along minor drainage lines within South Hedland and around Wedgefield in the 1 in 100 AEP event, but it is not anticipated that there would be any over-floor flooding of buildings within either town under existing conditions. Figure 4-6 shows the pattern of velocities under existing conditions for the 1 in 100 AEP flood event, with velocities exceeding 1.0 m/s in central channels.

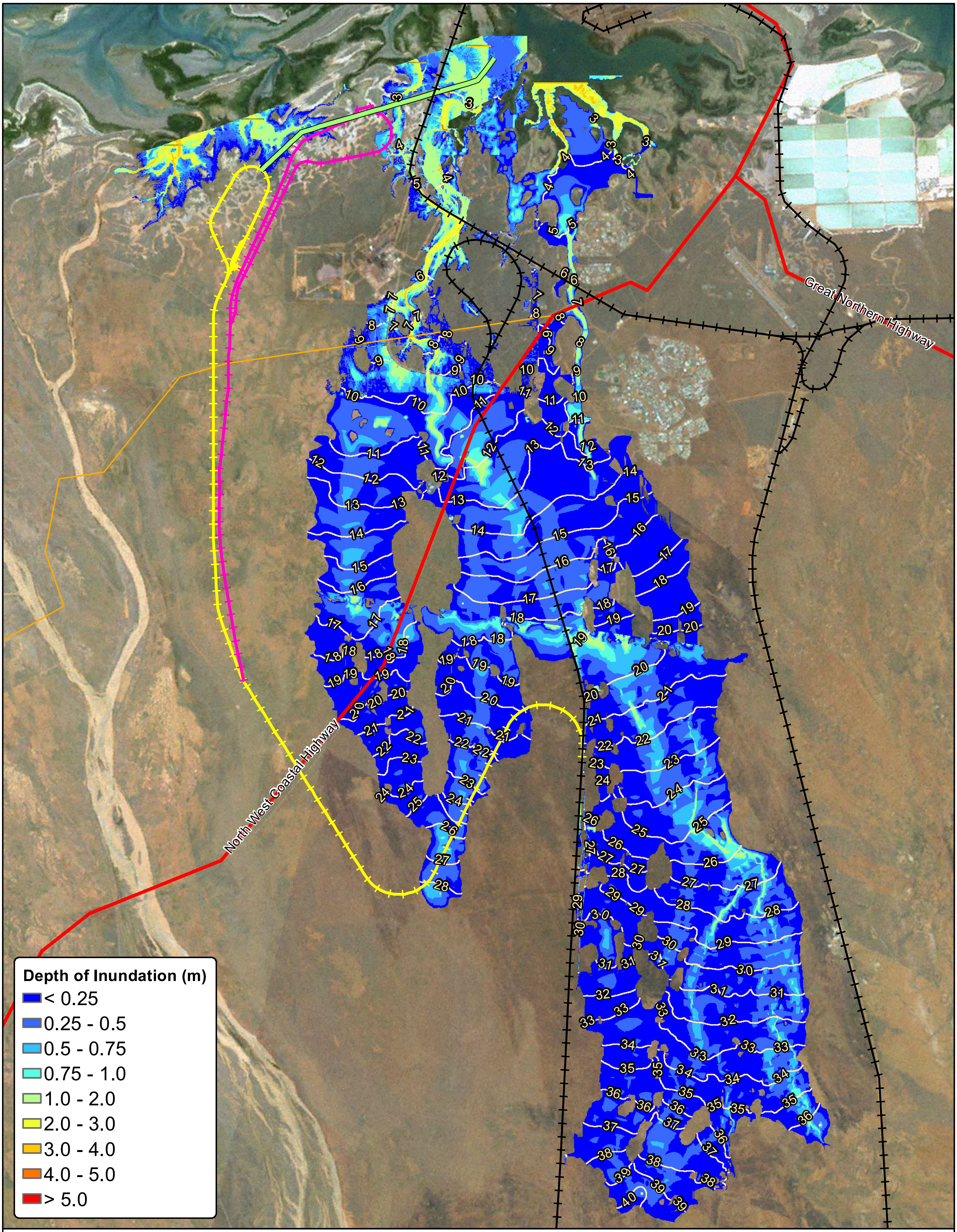

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Figure 4-1 Existing Case 10 year ARI Peak Inundation and Peak Water Level

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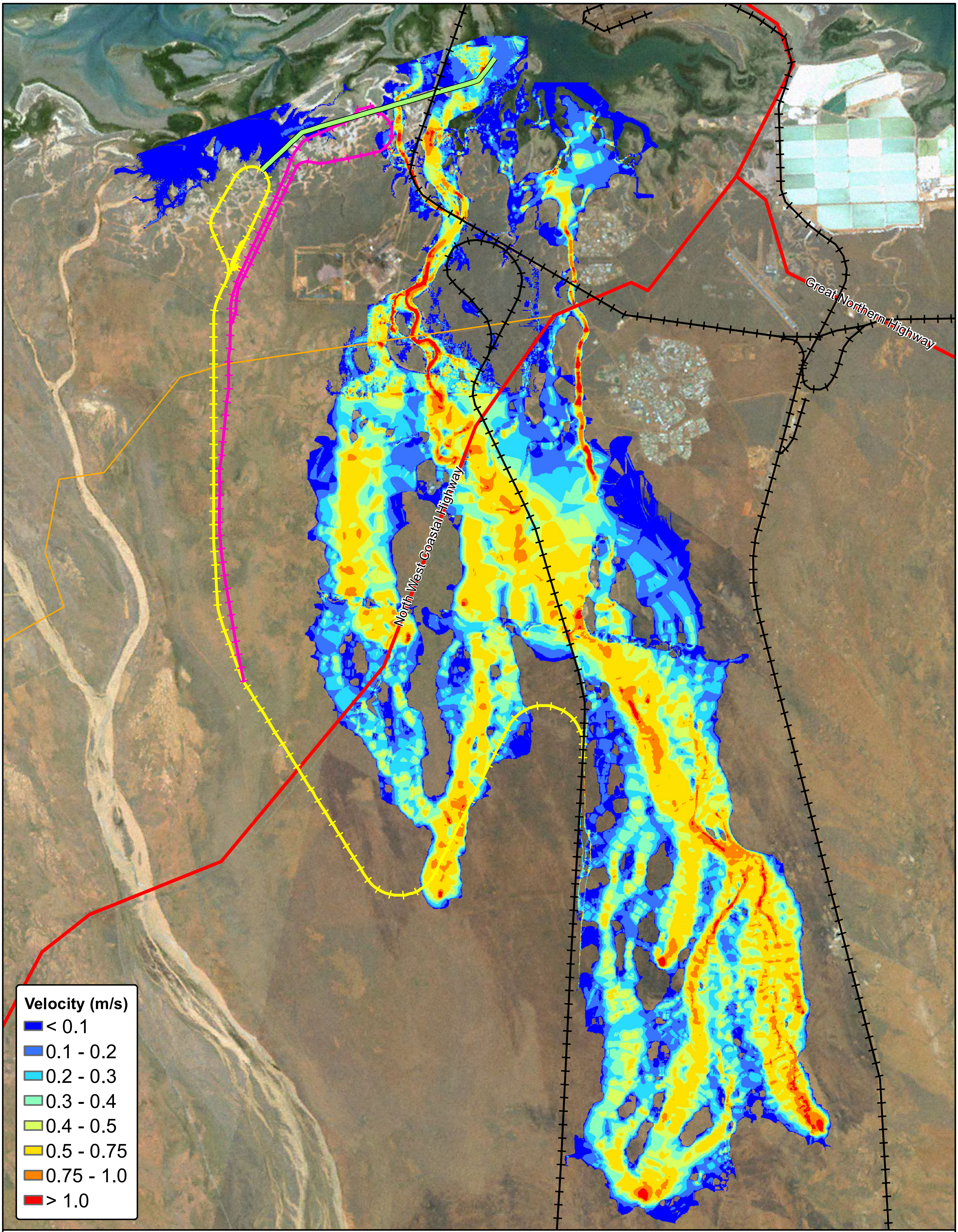

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-2 Existing Case 10 year ARI Peak Velocity

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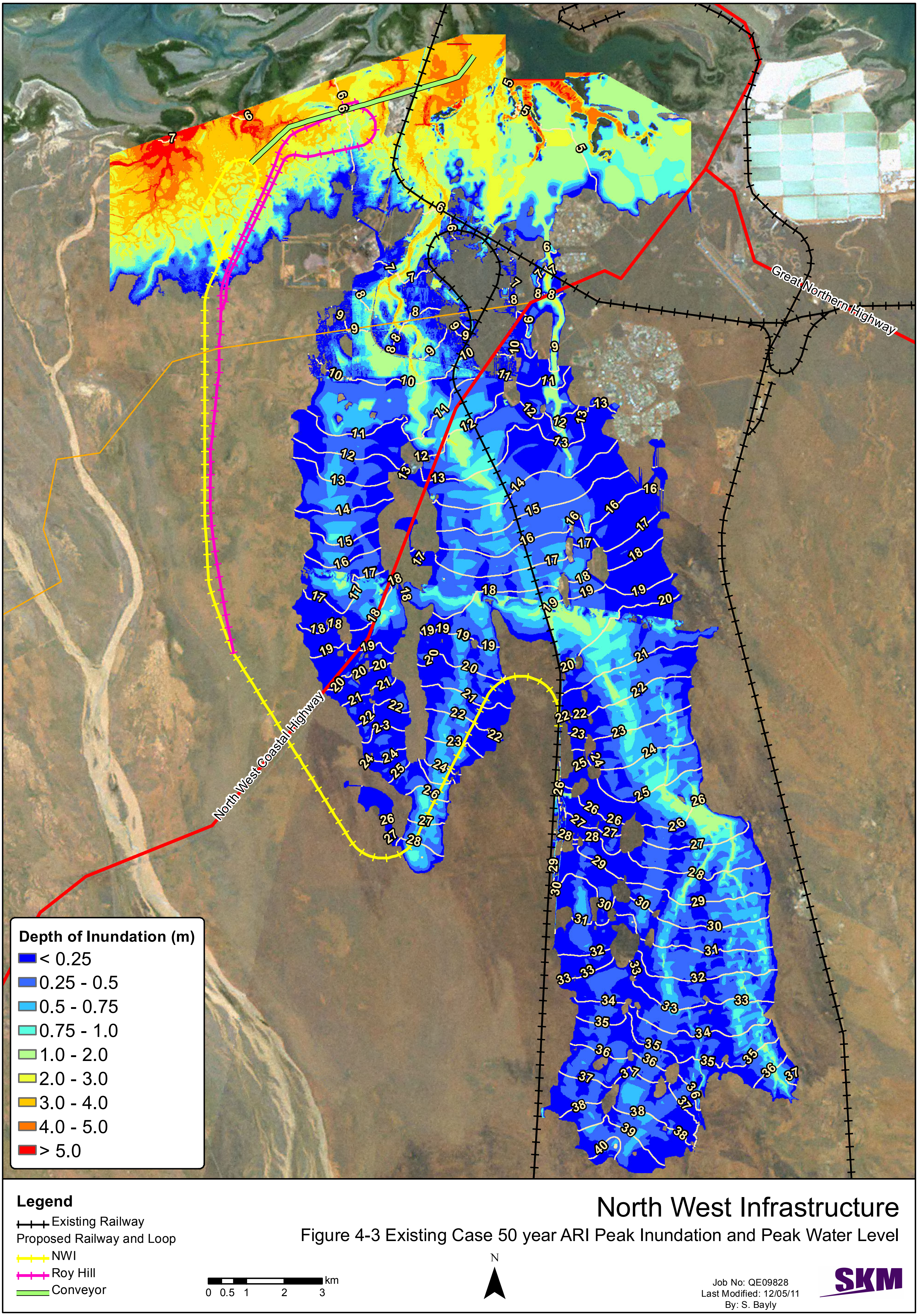

North West Infrastructure

Figure 4-3 Existing Case 50 year ARI Peak Inundation and Peak Water Level

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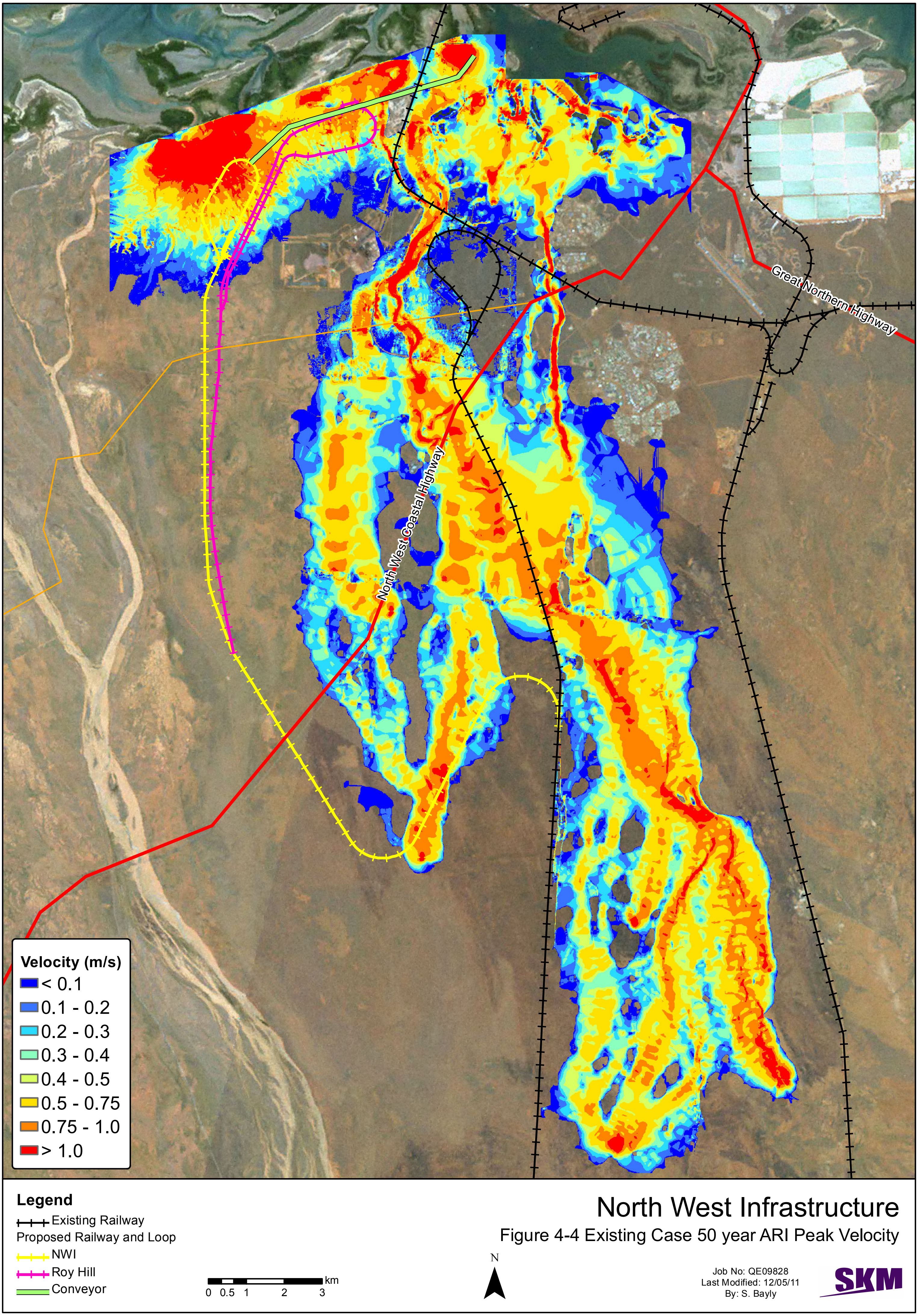

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Figure 4-4 Existing Case 50 year ARI Peak Velocity

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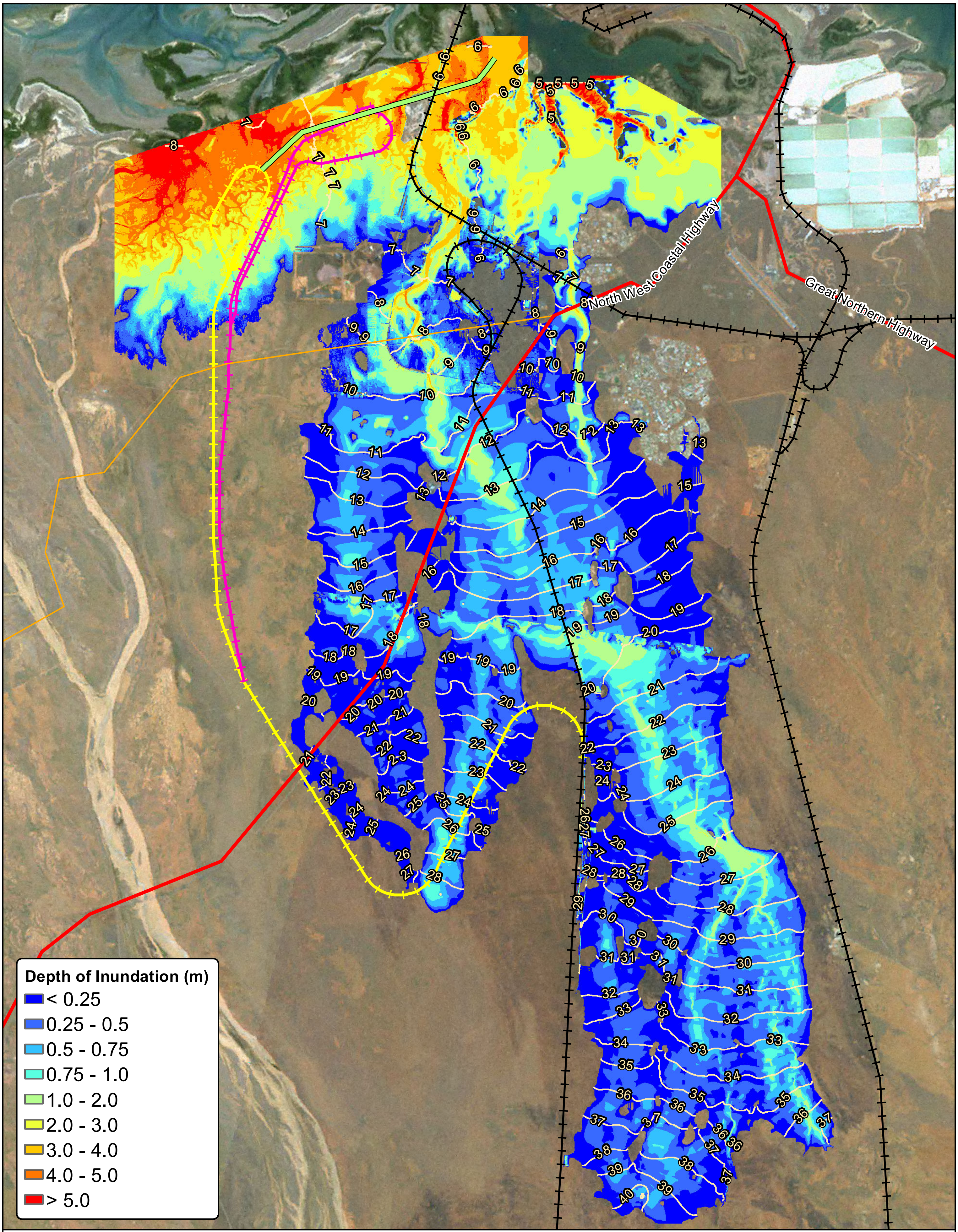

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-5 Existing Case 100 year ARI Peak Inundation and Peak Water Level

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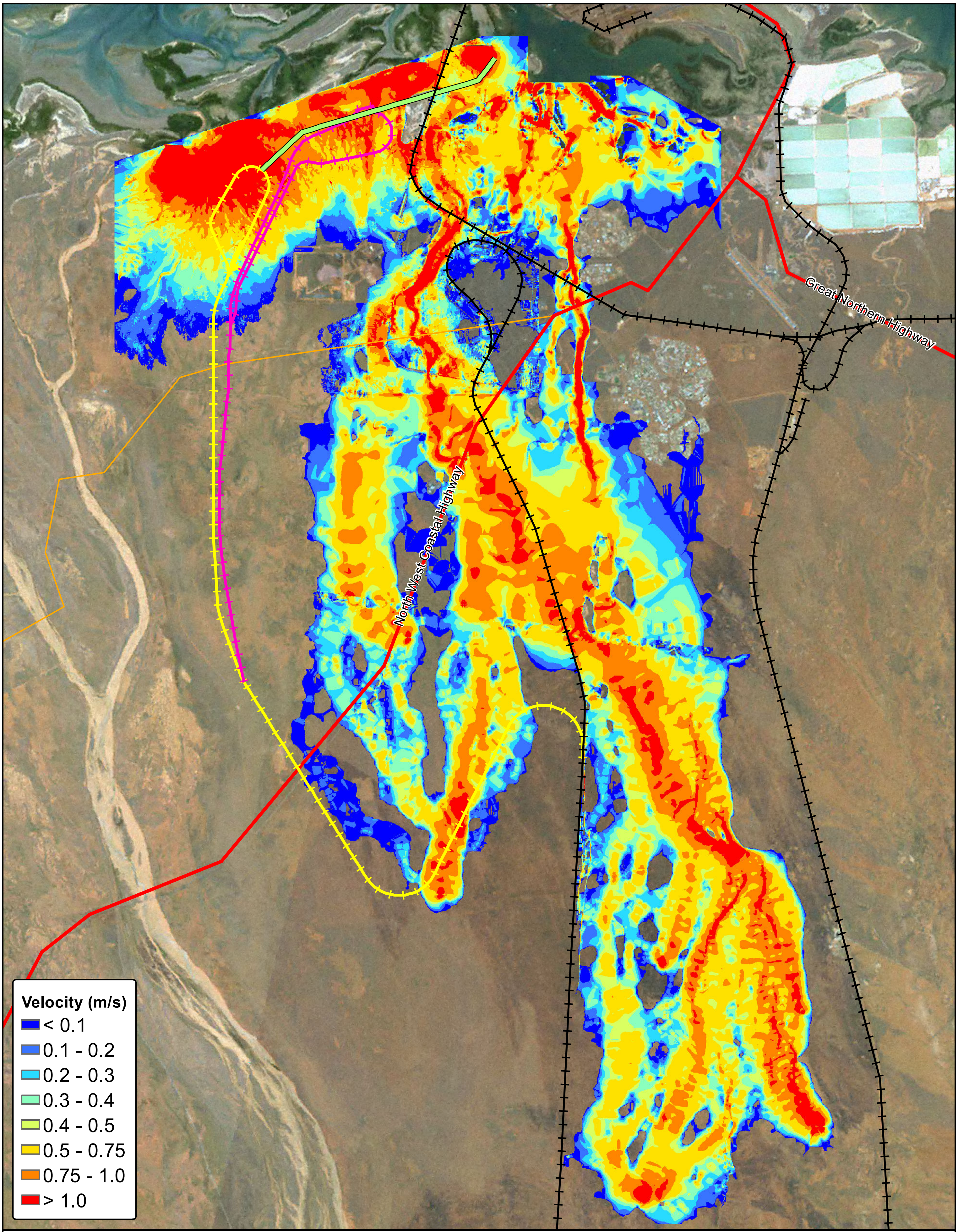

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-6 Existing Case 100 year ARI Peak Velocity

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4.3 Developed Case Flood Modelling

4.3.1 Model Development

The developed case model was derived from the model for the existing case, with changes to the terrain and structures to represent the railway loop and stockyard, adopted railway spur line and the conveyor embankment. The model terrain was modified to increase levels for each of these features in the proposed development case to well above the 1 in 100 AEP flood level. In addition, waterway openings were introduced into the adopted railway spur line and the conveyor embankment to allow for the passage of flood and storm surge flows. An embankment on the eastern side of South West Creek, upstream of the adopted railway spur line crossing of South West Creek, was introduced to direct flows through the culvert crossing by the spur line of South West Creek. In order to contain and direct the 1 in 100 AEP flood flows along South West Creek, the embankment was required to extend for 1.6 km in a south easterly direction away from the railway embankment to the north western side of the culvert.

4.3.2 Impact Assessment

The model was run for the 1 in 10, 1 in 50 and 1 in 100 AEP flood events including the proposed NWI and Roy Hill developments, with the proposed mitigation measures of the culverts where the proposed adopted railway spur line crosses South West Creek, the training embankment extending upstream of the South West Creek culvert crossing for 1.6 km in a south easterly direction and with the conveyors for both the NWI and Roy Hill developments supported on pylons.

The pattern of flooding for the 1 in 10 AEP event with the proposed NWI and Roy Hill developments is very similar to the pattern of flooding under existing conditions, as can be seen by comparing Figure 4-7 to Figure 4-1. With the proposed NWI and Roy Hill developments, both Wedgefield and South Hedland remain unaffected by the 1 in 10 AEP flood event. The afflux map for the 1 in 10 AEP event (Figure 4-8) shows that flood levels are projected to increase by up to 0.3 m where the proposed railway connection line to the FMG line would cross South West Creek and that the projected increase in flood levels dissipates to the point that there is no afflux for all locations more than 5 km downstream of the culvert crossing. No existing infrastructure would be affected by the increase in flood levels for the 1 in 10 AEP event that would result from the proposed developments. Figure 4-9 shows that flow velocities for the 1 in 10 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek.

The proposed railway loops for NWI and Roy Hill impede the passage of storm surge for the 1 in 50 and 1 in 100 AEP storm surge events. As a result, the affluxes in the 1 in 50 AEP event (as shown in Figure 4-11) show reductions in peak flood levels of more than 0.15 metres across the area to the south of the proposed Roy Hill railway loop and to the west of the BHP Billiton railway line and reductions of peak flood levels of about 0.1 metres in the area to the east of the BHP

Billiton railway line. The only areas where flood levels are projected to increase with the proposed NWI and Roy Hill developments are from the proposed culvert of the adopted railway spur line across South West Creek, for a distance of approximately 7 km downstream. No existing infrastructure would be affected by the increase in flood levels for the 1 in 50 AEP event that would result from the proposed developments. Figure 4-10 shows that with the proposed NWI and Roy Hill developments, the towns of Wedgefield and South Hedland remain above the 1 in 50 AEP flood event. Figure 4-12 shows that flow velocities for the 1 in 50 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek.

The proposed railway loops for NWI and Roy Hill impede the passage of storm surge for the 1 in 100 AEP storm surge events, such that there are widespread areas of reductions in peak storm surge levels to the south and east of the two proposed rail loops (as shown in Figure 4-14). The only areas where flood levels for the 1 in 100 AEP event are projected to increase with the proposed NWI and Roy Hill developments are from the proposed culvert of the adopted railway spur line across South West Creek, for a distance of approximately 7 km downstream. No existing infrastructure would be affected by the increase in flood levels for the 1 in 100AEP event that would result from the proposed developments. Figure 4-13 shows that with the proposed NWI and Roy Hill developments, the towns of Wedgefield and South Hedland remain above the 1 in 100 AEP flood event.

Figure 4-15 shows that flow velocities for the 1 in 100 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek. For the 1 in 100 event with the proposed developments, the peak velocities reach 2.4 m/s near to the eastern end of the proposed Roy Hill railway loop. Roy Hill propose to install a revetment wall in this location to withstand velocities and flows from flooding and storm surge events. Peak velocities in the 1 in 100 AEP event near to the western edge of the existing FMG railway loop reach 1.2 m/s, which is the same as velocities at this location for the 1 in 100 event under existing conditions. Velocities in the 1 in 100 AEP event under with the proposed developments would exceed 1 m/s for a distance of approximately 1 km downstream of the proposed culvert crossing of South West Creek, with a maximum velocity of 2.0 m/s immediately downstream of the culvert.

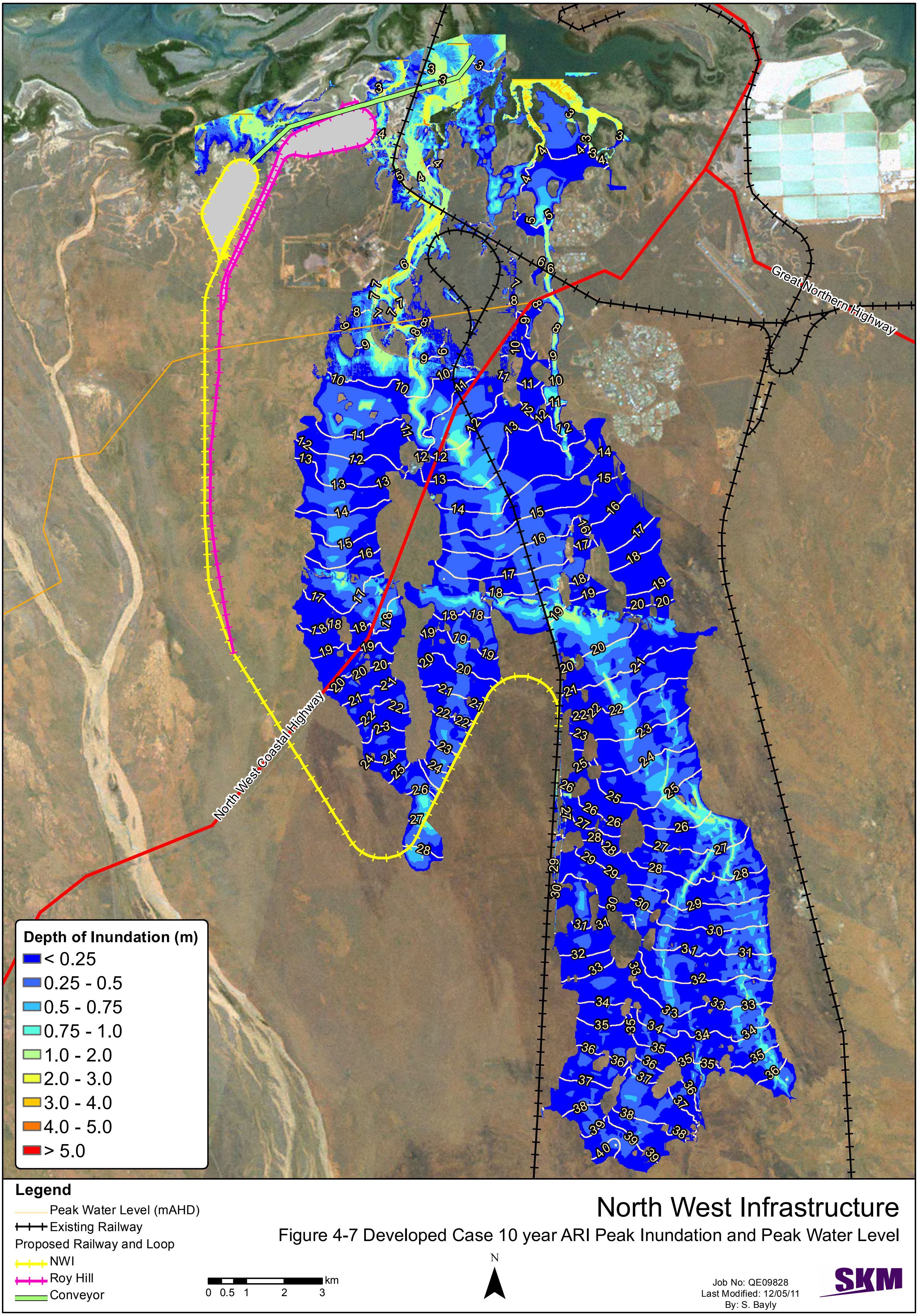

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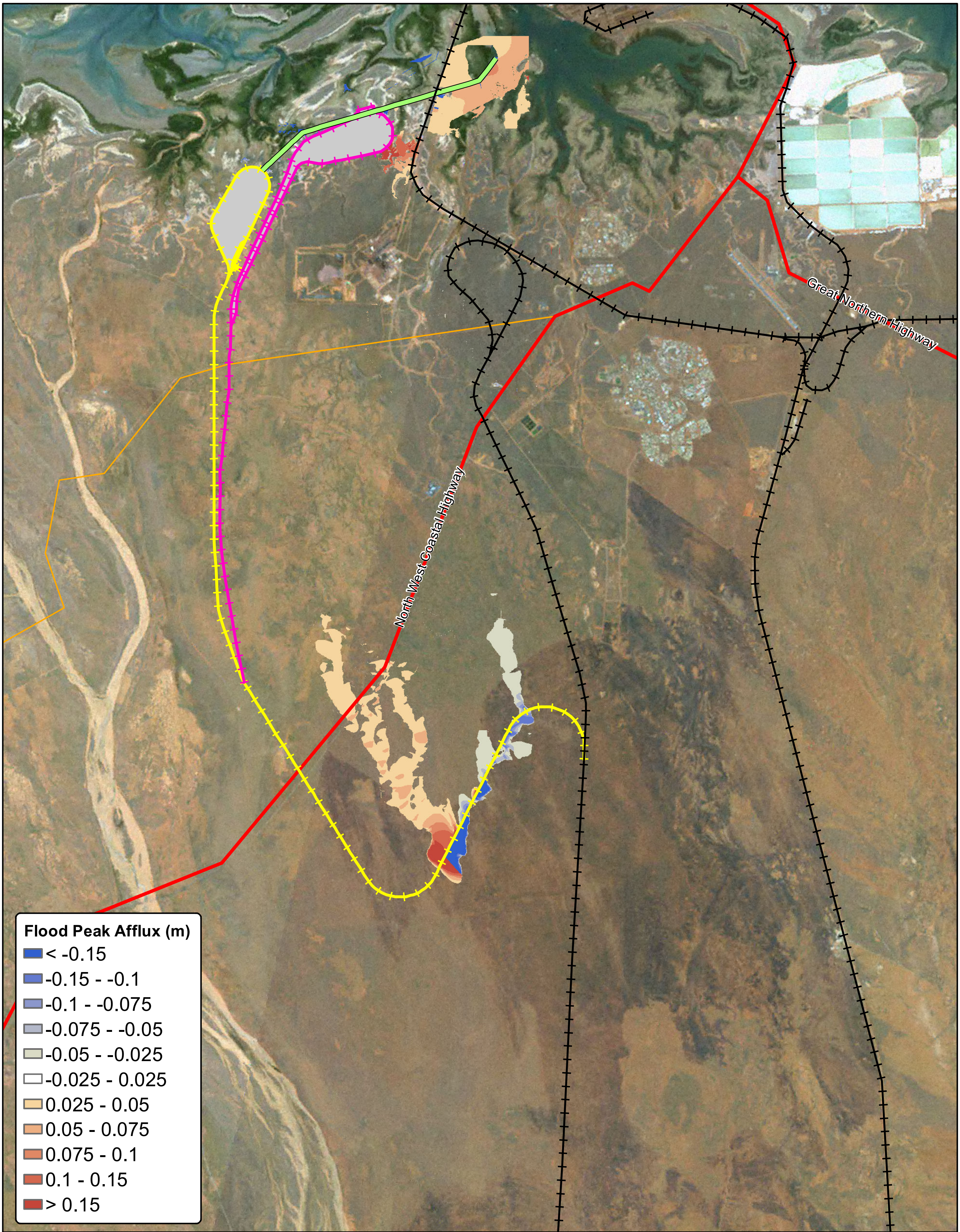
Figure 4-7 Developed Case 10 year ARI Peak Inundation and Peak Water Level



Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

km
0 0.5 1 2 3

N

North West Infrastructure

Figure 4-8 Developed Case 10 year ARI Afflux

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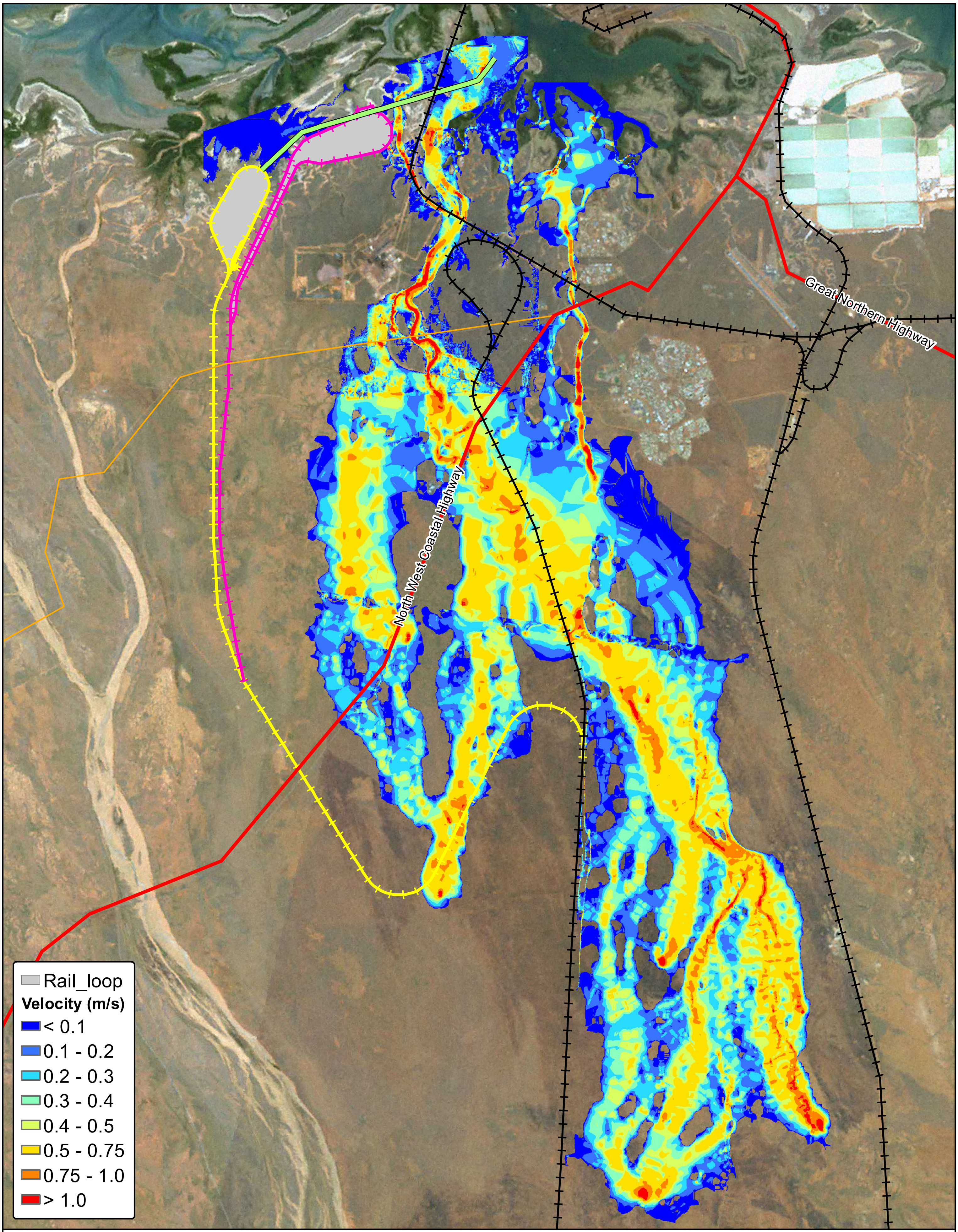

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-9 Developed Case 10 year ARI Peak Velocity

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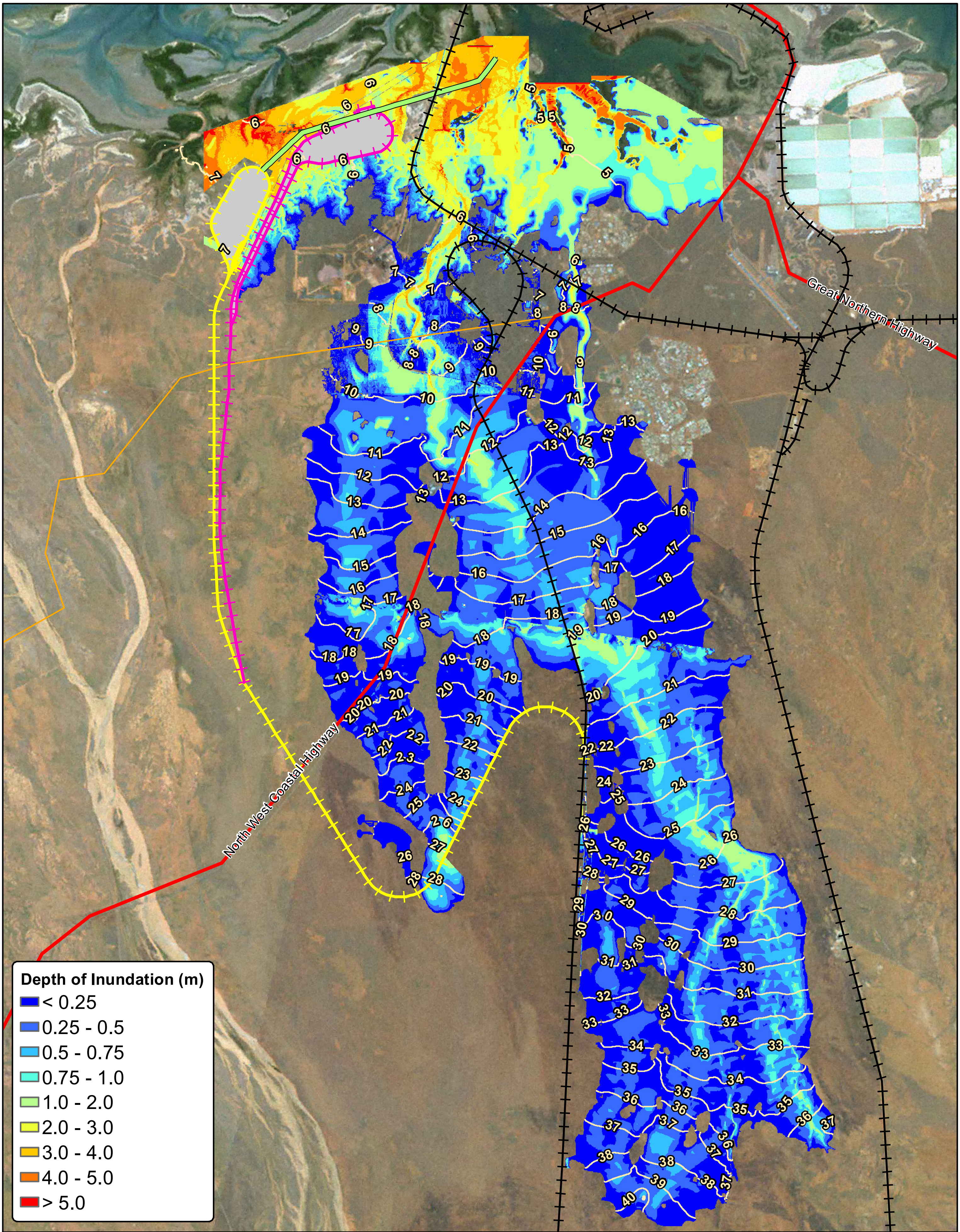

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

North West Infrastructure

Figure 4-10 Developed Case 50 year ARI Peak Inundation and Peak Water Level

N

0 0.5 1 2 3 km

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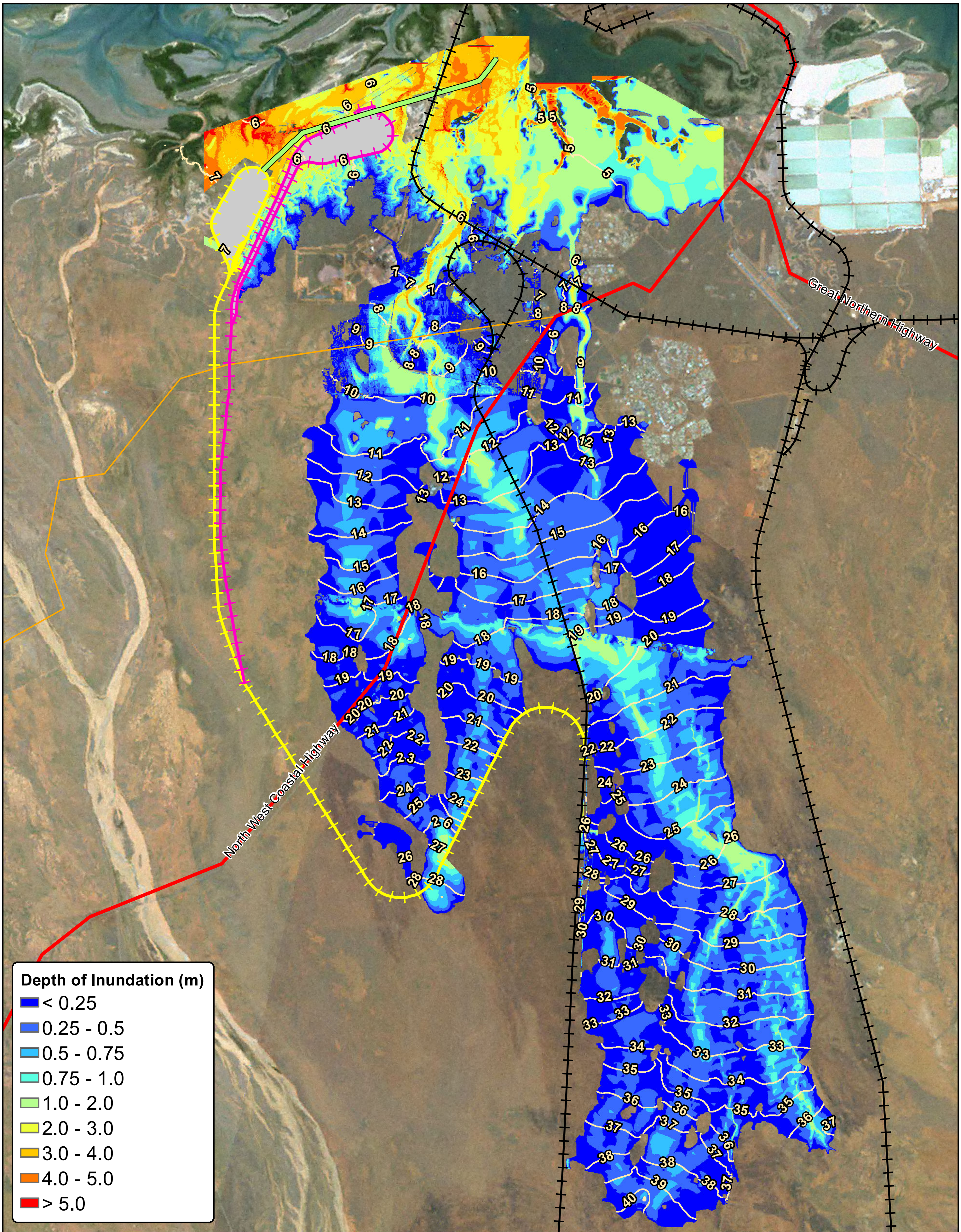

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-10 Developed Case 50 year ARI Peak Inundation and Peak Water Level

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By: S. Bayly

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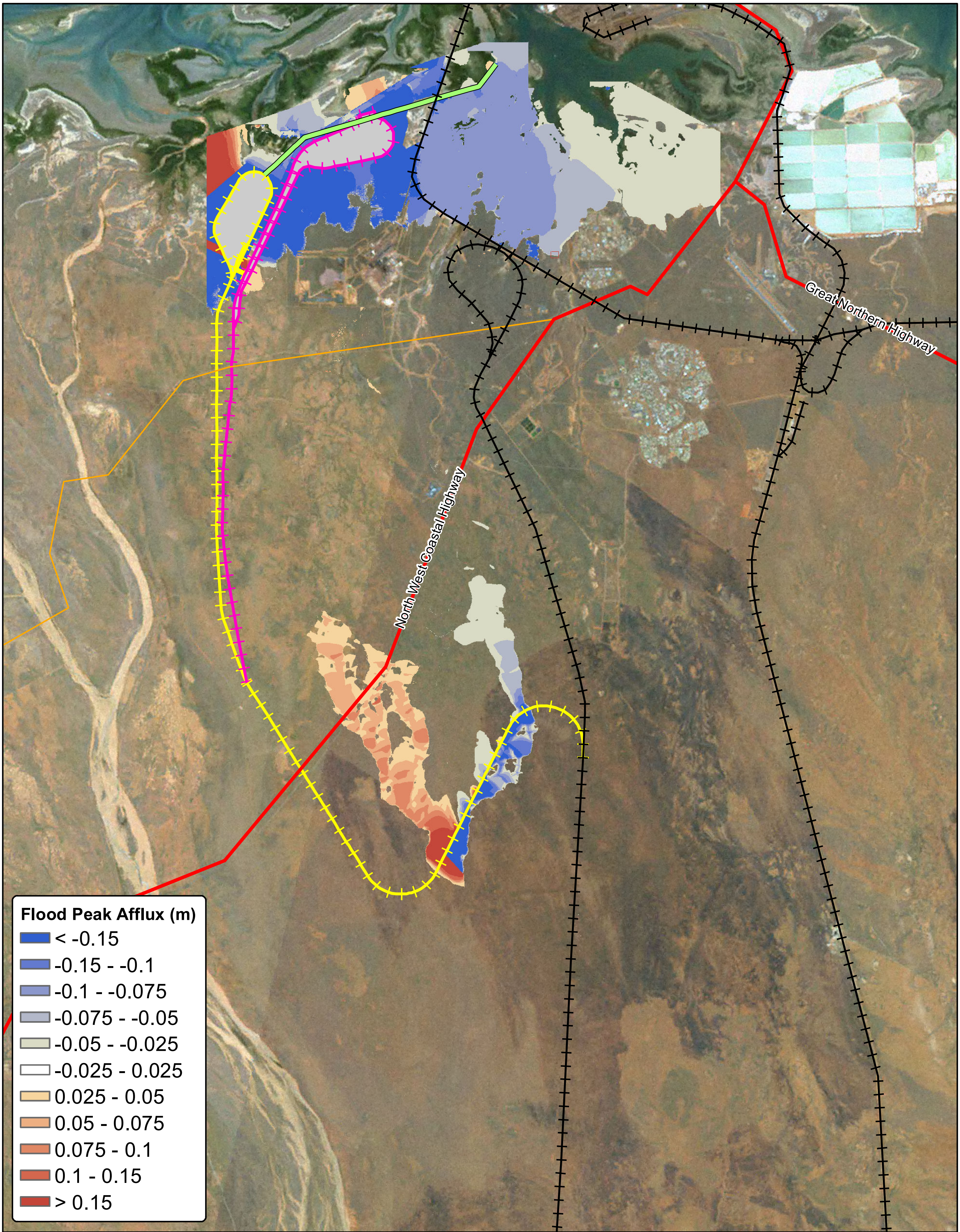

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-11 Developed Case 50 year ARI Afflux

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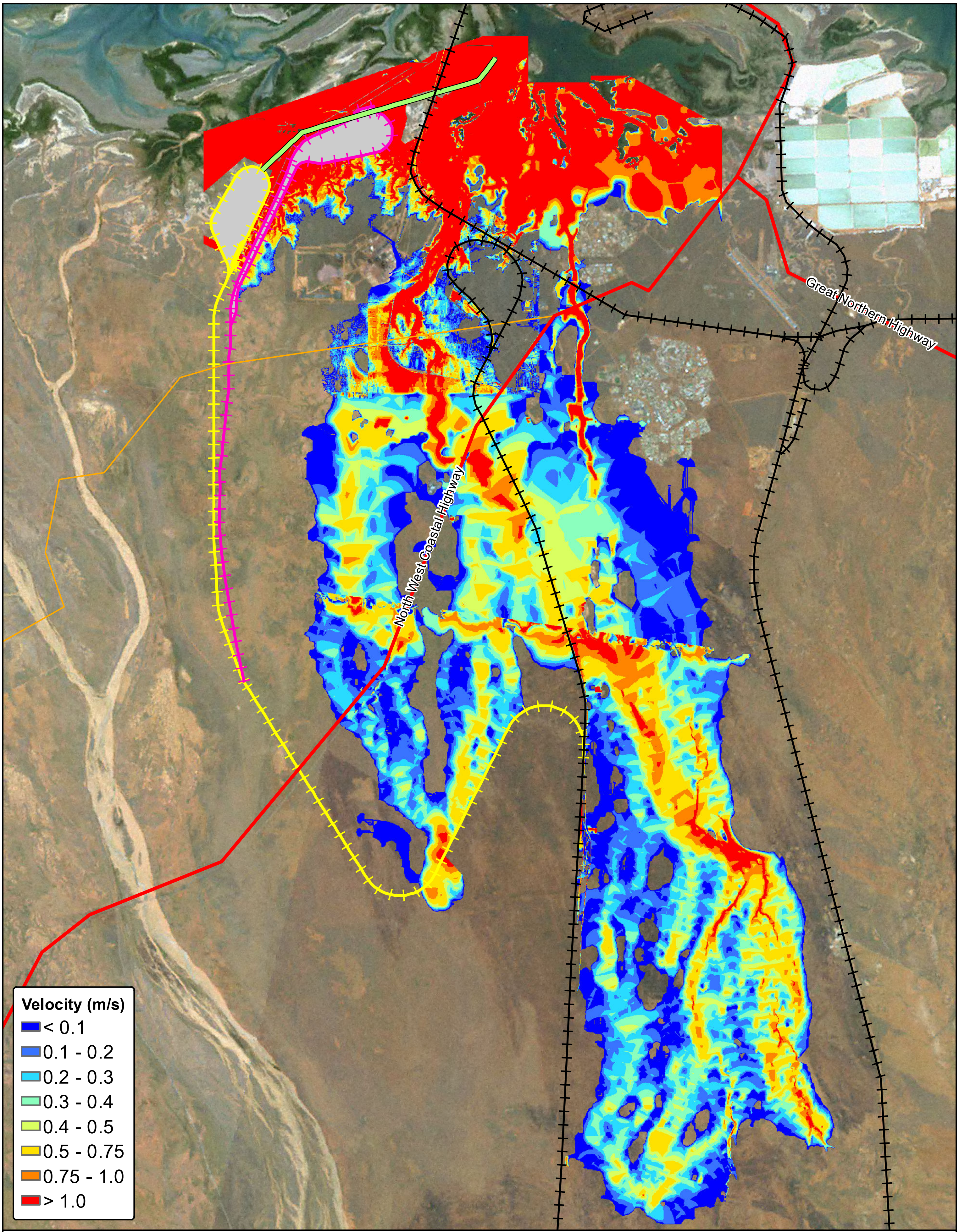

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-12 Developed Case 50 year ARI Peak Velocity

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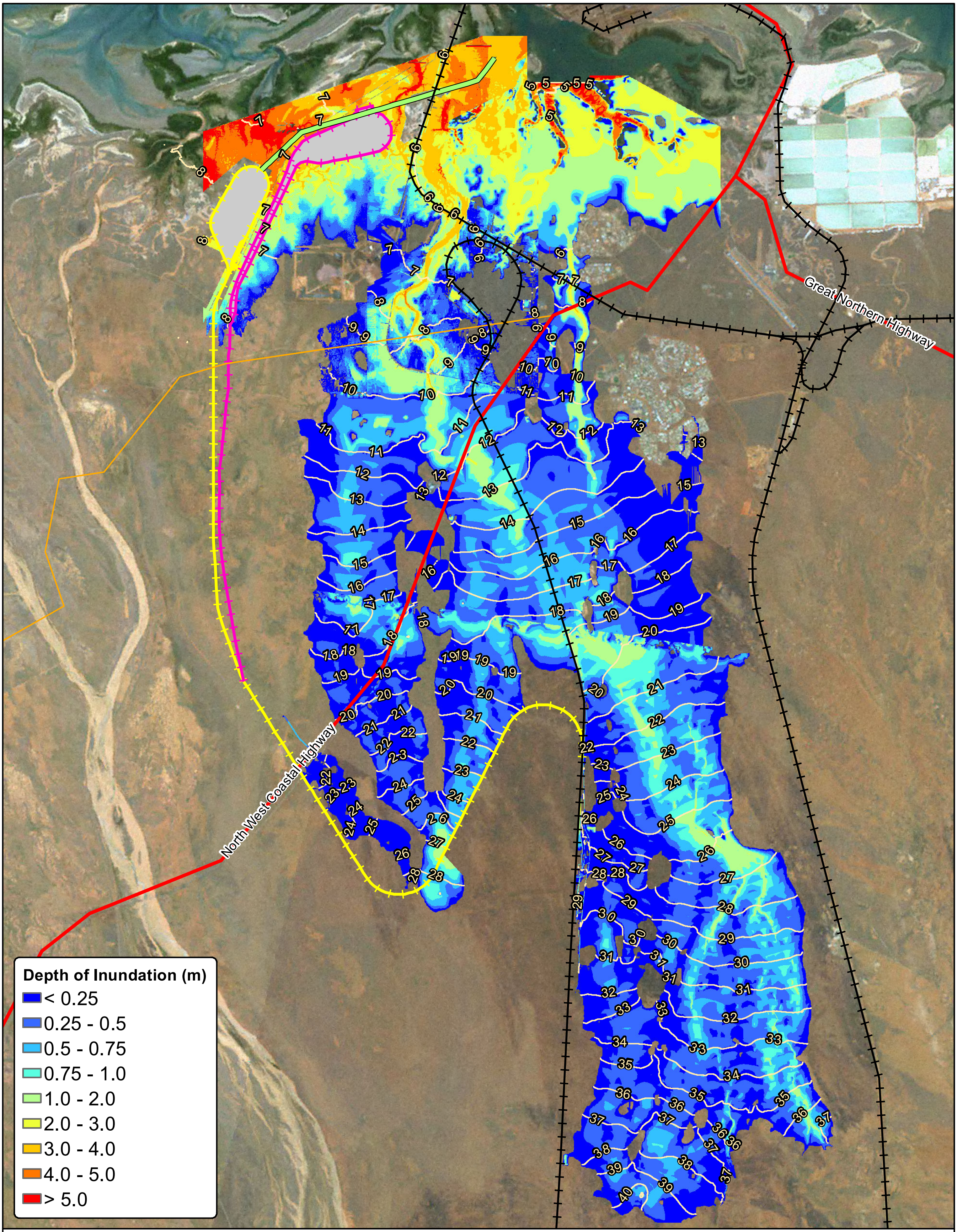

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-13 Developed Case 100 year ARI Peak Inundation and Peak Water Level

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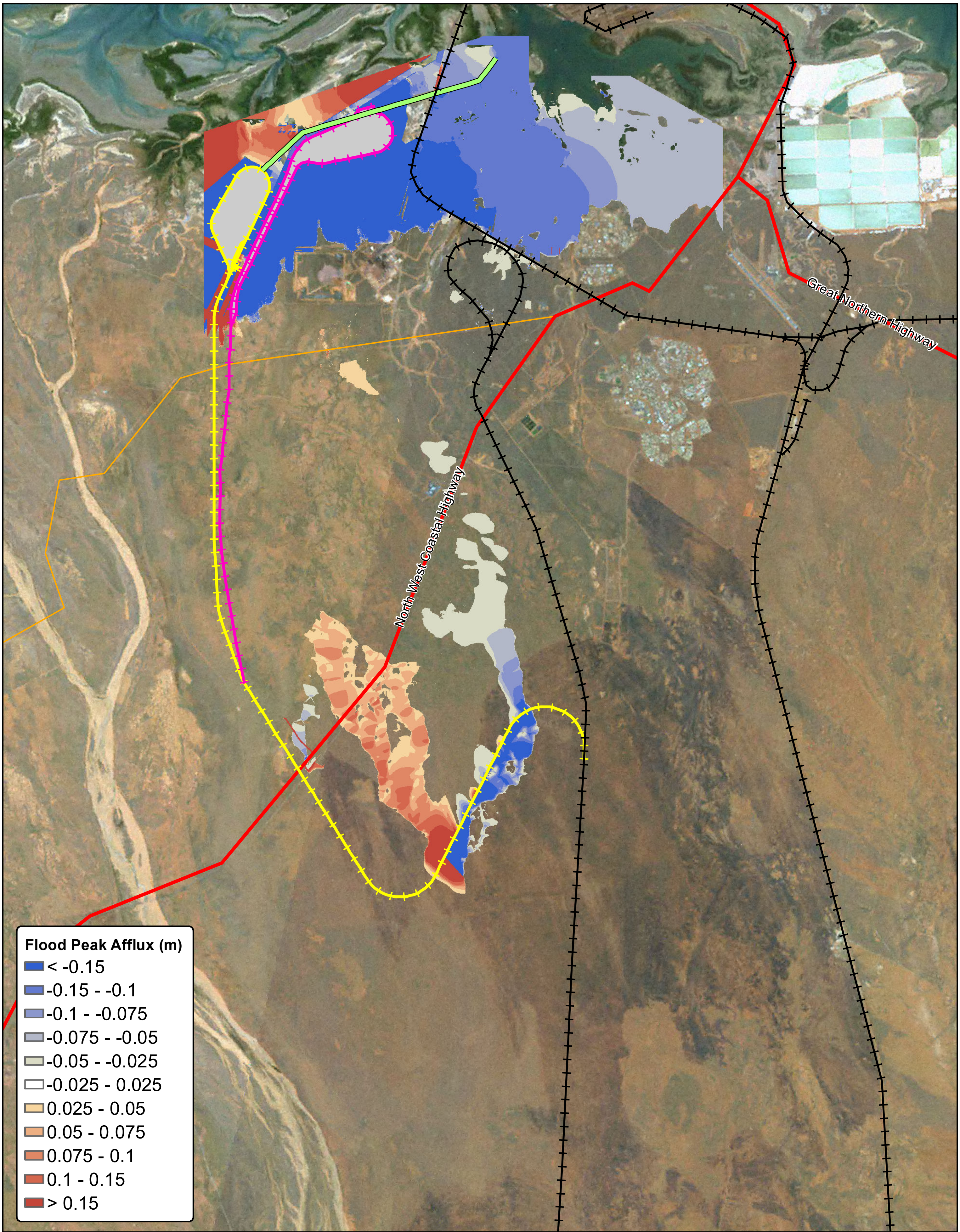

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

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Figure 4-14 Developed Case 100 year ARI Afflux

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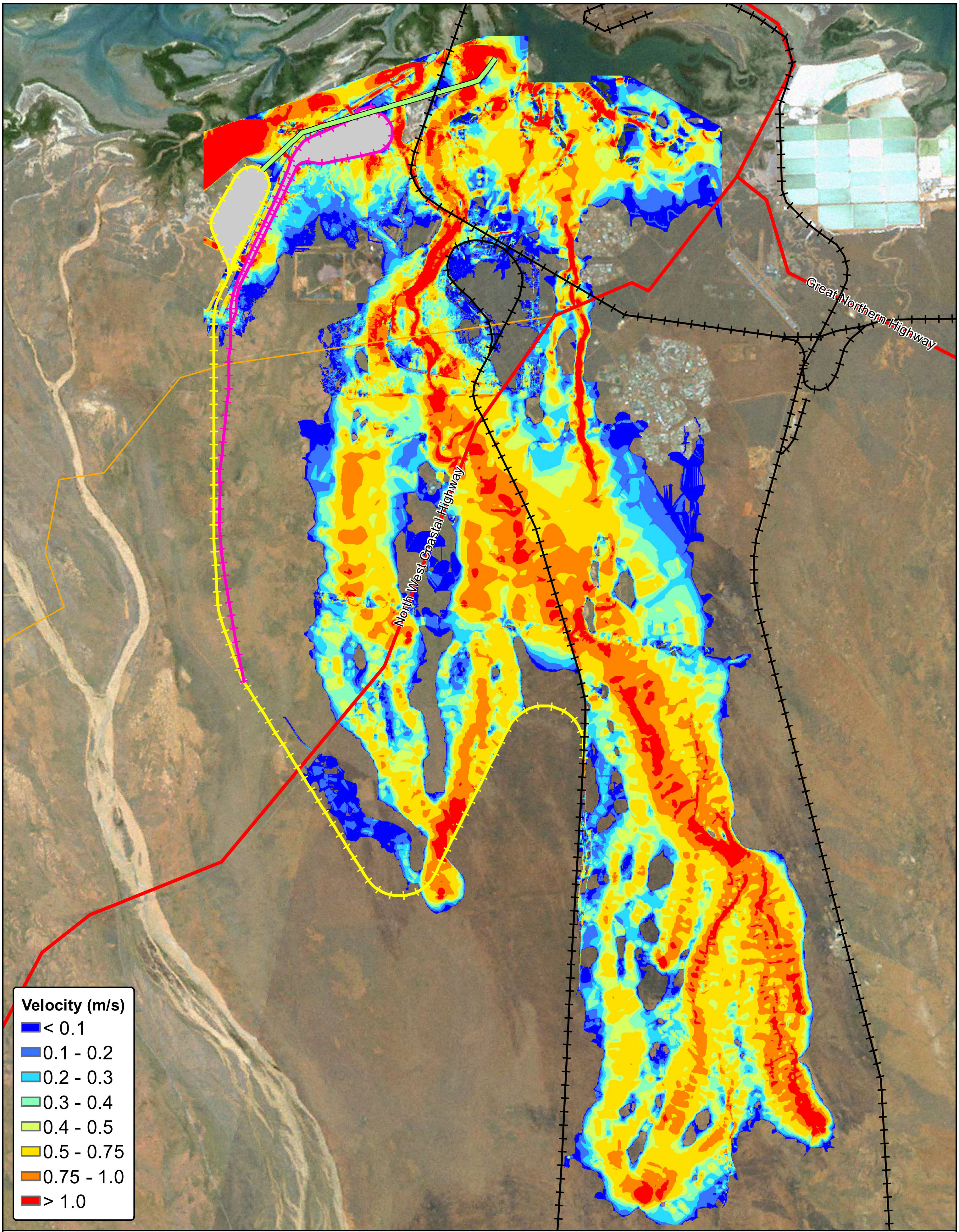

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-15 Developed Case 100 year ARI Peak Velocity

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5 Assessment of Potential Surface Water Impacts

5.1 Affects on Floodplain Storage

Where the footprint of the railway loop intersects with areas of potential flood inundation under existing conditions, there will be a resulting loss of floodplain storage. The footprint of the NWI railway loop and facilities is 0.15 km², which results in a loss of up to 0.15 km² in floodplain storage. The proposed Roy Hill development would consume a similar area of floodplain. However, most of the proposed railway lines for both developments would be located on more elevated land along the ridge between the South West Creek and Turner River catchments, and therefore much of this floodplain area would only be inundated during a flood event that is more extreme (lower probability) than the 1 in 100 AEP event. The impact of the loss of floodplain storage is considered in conjunction with the effects of impedance to surface flow represented in the hydrological and hydraulic modelling.

5.2 Changes to Rainfall Runoff Rates

Installation of the rail loop, car dumper, access roads, stockyard for iron ore, conveyor, two berth wharf and ancillary infrastructure buildings will cause increases in runoff rates from those that would be observed under the natural catchment conditions, due to increases in the impervious area. This represents 0.03% of the total combined catchment area of South and South West Creeks. This change in the volume of runoff produced is insignificant when compared with the overall total volume of runoff generated from the existing catchments of South and South West Creeks during flood events.

During minor rainfall events there will be additional runoff generated from these impervious areas that would otherwise runoff into areas surrounding the project site. Stormwater management practices, such as detention of runoff from impervious areas to reduce flow velocities and the prospects of scour, may be required to manage these local increases in runoff rates.

5.3 Impedance of Surface Flow

The modelling simulations demonstrated that the largest impact upon runoff is the impedance of surface water flow. If no mitigation measures were put in place, the adopted railway spur line, rail loop and conveyor embankment would constrain the movement of surface water flows.

The spur line crosses South West Creek, which if unmitigated would cause flows from South West Creek to back up and pond along the upstream side of the adopted railway spur line. The mitigation measures to avoid this are to provide adequate culvert capacity through the South West Creek crossing for the adopted railway spur line and to place contour embankments along the upstream side of the adopted railway spur line and to the North of the South West Creek culvert crossing to force South West Creek flows underneath the railway at the crossing location.

If the conveyor were built entirely along an embankment, with no waterway openings, this would impede both flood runoff from South and South West Creeks flowing out to the ocean and storm surge penetrating inland from the ocean. The mitigation measure to maintain impedance of surface water flows at acceptable levels is to build the conveyor structure on pylons to allow for flood and storm surge flows to pass underneath the conveyor.

The proposed development, even with mitigation measures in place, will modify the velocity of flows in the floodplain of South and South West Creeks during flood events, as documented above. Scour protection may be required, particularly in the vicinity of the waterway openings in the conveyor and the culvert through the adopted railway spur line, to control erosion during flood and storm surge events.

5.4 Impacts on Surface Water Quality

Rainfall on iron ore stockpiles and impervious surfaces around the rail loop and conveyor will cause runoff that may contain sediment and low levels of other contaminants. Silt trap and sedimentation basin facilities will be developed to trap sediments washed off the stockpile areas during rainfall events to prevent these materials flowing unmitigated into the receiving environment.

Modelling results indicate the velocities of flows in the floodplain around the site may change as a result of the development, although due to the relatively low level of encroachment of the project site into the floodplain these changes in velocity are expected to be small. Suspended sediment loads of downstream water may be increased if alteration of flow regimes results in scour velocities created downstream of the development. Areas where high flow velocities may be expected should be identified and appropriate measures taken to slow flow and contain sediments on site.

6 Management Measures

During rainfall events, some additional stormwater runoff will be generated from new impervious areas of the of the rail loop, car dumper, access roads, stockyard for iron ore, conveyor, two berth wharf and ancillary infrastructure buildings. On-site stormwater management measures may need to be implemented to detain the runoff produced from these areas and to minimise scour caused by direct runoff from these areas. A review of the DFS indicates a detention basin of 10.3 ha surface area is incorporated for management of runoff within the stockpile area.

The stockpiles, car dumper and conveyor loader will be located above the 1 in 100 AEP flood and storm surge level, to minimise the probability of this infrastructure being inundated. In the event of a large flood or storm surge event that floods the rotary car dumper and the facilities for detaining water released during unloading of the rail cars, some sediment and other contaminants may be released into the environment. During a large flood event (rarer than 1 in 100 AEP), there would be large volumes of water inundating the flood plains of South and South West Creek so that there would be considerable dilution of the potentially contaminated water. Volumes of flood water would be sufficiently large that there would not be expected to be appreciable increases in the concentrations of contaminants.

Culverts will be designed to allow flood flows to pass through the adopted railway spur line at the South West Creek crossing during the 1 in 100 AEP event without causing inundation of the railway line. A regular programme of inspection and maintenance of the culverts would be implemented so that they continue to perform their function in passing flows during flood events. Waterway openings, in the form of bridges or culverts, will be designed in the conveyor embankment to pass the 1 in 100 AEP flood and storm surge event.

Baseline and continuous monitoring of sediment and other pollutants during construction and operation of the facility will be conducted to detect any changes in water quality due to these activities.

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North West Infrastructure Proposed Multi-User Iron Ore Export Facility

SURFACE WATER IMPACT ASSESSMENT COMPONENT OF ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT

- QE09828-EV-RP-0001-RevisionC
- 8 June 2011

Proposed Multi-User Iron Ore Export Facility

SURFACE WATER IMPACT ASSESSMENT COMPONENT OF ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT

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Executive Summary

Background

North West Infrastructure (NWI) has proposed a development of a multi-user iron ore export facility for Port Headland in Western Australia. The *Landside* project as described within the draft definitive feasibility study (DFS) comprises a wharf, overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the northern portion (approximately 5km) of the western rail spur. The Landside project ends at the southern boundary of the land proposed to be vested in the PHPA, and mirrors a similar infrastructure development by the Roy Hill Iron Ore (RHIO) recently approved by the Environmental Protection Authority (EPA 2011).

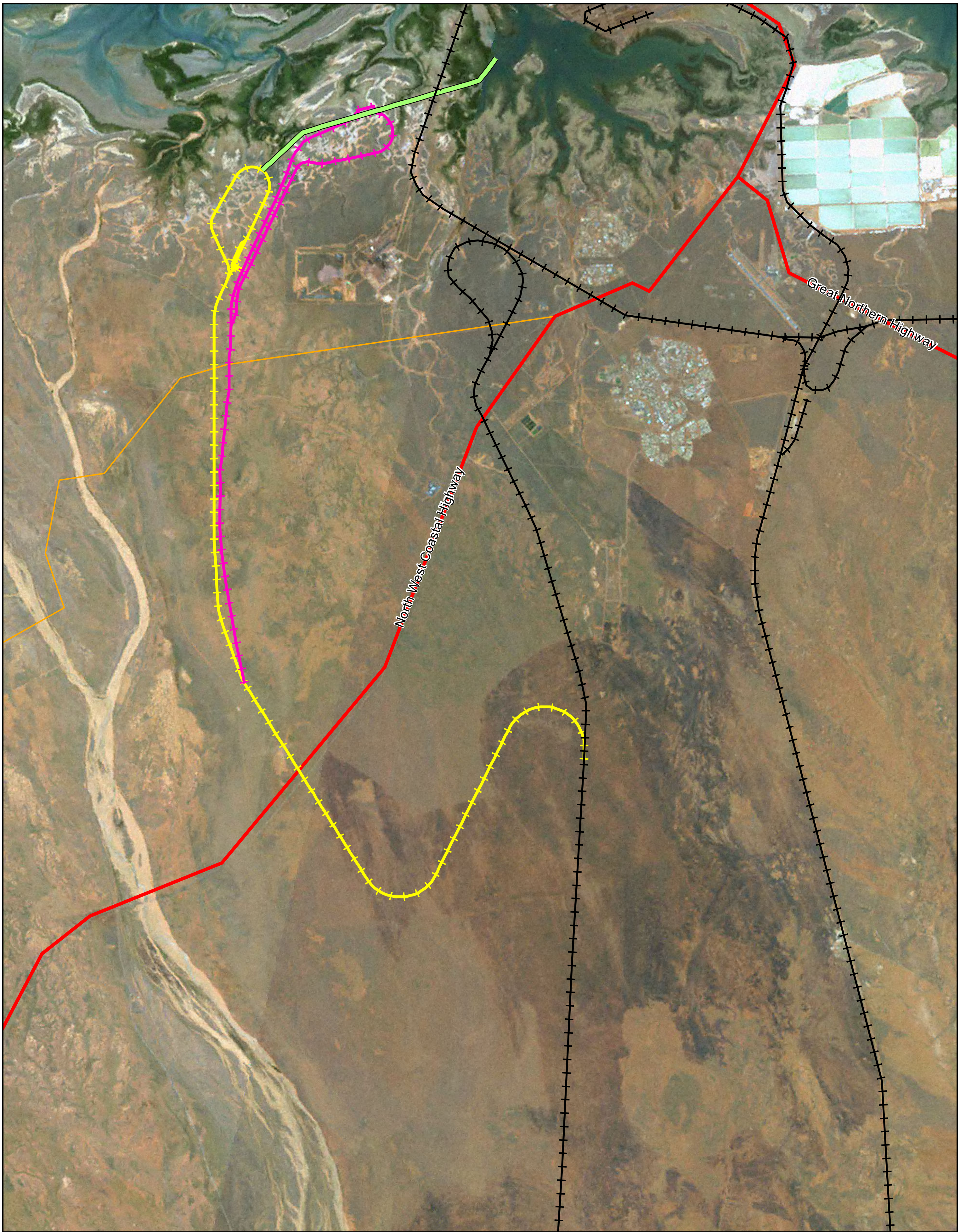
The *Rail* project involves a rail connection between the western rail spur and conections to any combination of rail lines operated or proposed by and the Fortescue Metals Group (FMG), BHP-Billeton Iron Ore (BHP-BIO) and RHIO rail lines. The alignment of existing infrastructure and the proposed NWI facilities are shown in Figure 1-1, with the FMG connection identified as a reasonable worst case and adopted as the base case for modelling in this report.

The proposed NWI rail loop, conveyor and port infrastructure is located immediately to the west of a similar railway loop, railway line, iron ore stockpile, conveyor and wharf facility that has been proposed by RHIO (Roy Hill Infrastructure Pty Ltd, 2010), as shown in Figure 1-1.

Floods and storm surges are known to occur in the vicinity of the project site. Such events may result in inundation of some of the proposed infrastructure and modifying the direction, depth and velocity of flows in areas around the project site. As shown in Figure 1-1, the proposed railway connecting line from the FMG line crosses the drainage line of South West Creek. The NWI and Roy Hill lines split after crossing South West Creek and travel parallel to one another and they also run parallel to the major drainage lines of South West Creek, South Creek and the Turner River.

The Project Area is located within the catchments of South and South West Creeks on the Pilbara Coast near Port Hedland. The Project area is in the Western part of this area and part of the indicated NWI railway alignment runs along the divide between the catchments of South West Creek and the Turner River.

South and South West Creeks drain into Port Hedland Harbour, which then drain into the Indian Ocean. The catchments of South and South West Creeks are sufficiently flat (particularly in the northern part of each catchment) that during periods of flood or high tides and storm surge the flows from the two creeks combine into a single drainage system. During flood periods flows from the catchment of South Creek can cross over and flow into South West Creek and vice versa. The combined catchment area of South and South West Creeks is 557 km^2.



Legend
Existing Railway
Proposed Railway and Loop
NWI
Roy Hill
Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 1-1 Locaility Plan

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The Turner River is a much larger catchment than the combined catchments of South and South West Creeks, having a total catchment area of 3556 km^2. The alignment of the proposed Project is such that no surface water flows are expected from the Project into the Turner River and that flows in the Turner River would not be modified by the Proposed Project.

Method

A combined hydrological and hydraulic model was used to investigate flooding under existing conditions and to assess impacts of the proposed development and also the combined impact of the NWI and Roy Hill developments. A hydrological model was established for the combined catchments of South and South West Creeks in the RORB rainfall runoff modelling program. Peak flows estimated from the RORB model were validated against flood peaks assessed from a regional frequency analysis of flood peaks recorded from other catchments of similar size in the Pilbara. A model of the area containing the Project site was created using a MIKE FLOOD dynamically coupled one-dimensional and two-dimensional hydraulic model.

Some parts of the Project site footprint would be expected to be inundated by storm surge and high tide, rainfall derived runoff in the catchments of South and South West Creeks or by a combination of storm surge and rainfall derived runoff. A Tropical Cyclone approaching or crossing the coast in the vicinity of Port Hedland is likely to lead to a combined storm surge and rainfall runoff flooding event in this area. The scenarios modelled to estimate peak flood levels were as shown in Table 1 Combined catchment runoff and storm surge events modelled.

- **Table 1 Combined catchment runoff and storm surge events modelled**

Nominal Overall AEP of Event	AEP of Creek Runoff Flood Event	Tidal and Storm Surge Boundary Condition
1 in 10 AEP	1 in 10 AEP	Mean High Water Spring Tide Level (constant head boundary)
1 in 50 AEP	1 in 50 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 50 AEP storm surge event, sinusoidally varying temporal pattern
1 in 100 AEP	1 in 100 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 100 AEP storm surge event, sinusoidally varying temporal pattern

The developed state was modelled by introducing the proposed NWI railway loop, conveyor alignment (embankment and trestle) and adopted railway spur line into the model. The developed state model also includes the proposed impact of the Roy Hill loop, conveyor and adopted railway spur line, so that the combined effects of both developments could be assessed.

Assessment of Flood Impacts

Even under existing conditions, combined storm surge and flood events with an AEP of 1 in 50 and 1 in 100 cause flooding of the floodplains of South and South West Creeks, although the towns of Wedgefield and South Hedland are largely above the 1 in 100 AEP flood level. There is some flooding along minor drainage lines within South Hedland and around Wedgefield in the 1 in 100 AEP event, but it is not anticipated that there would be any over-floor flooding of buildings within either town under existing conditions.

The modelling simulations demonstrated that the largest impact upon runoff is the impedance of surface water flow. If no mitigation measures were put in place, the NWI and RHIO railway spur lines, rail loops/ stockpiles and conveyors would constrain the movement of surface water flows.

The modelled NWI spur line runs approximately parallel to the RHIO alignment and crosses South West Creek. If unmitigated, rail embankments would cause flows from South West Creek to back up and pond along the upstream side of the alignment. The mitigation measures to avoid this are to provide adequate culvert capacity through the South West Creek crossing for the railway spur lines and to place contour embankments along the upstream side of the railway spur line and to the North of the South West Creek culvert crossing to force South West Creek flows underneath the railway at the crossing location.

If the conveyor were built entirely along an embankment, with no waterway openings, this would impede both flood runoff from South and South West Creeks flowing out to the ocean and storm surge penetrating inland from the ocean. Mitigation measures to sustain surface water flows at acceptable levels is to include culverts where embankments are to be used and elevate the conveyor structure on pylons to allow for flood and storm surge flows to pass underneath in critical flow areas.

The model was run for the 1 in 10, 1 in 50 and 1 in 100 AEP flood events including the proposed NWI and Roy Hill developments, with the proposed mitigation measures of the culverts where the adopted railway spur line crosses South West Creek, the training embankment extending upstream of the South West Creek culvert crossing for 1.6 km in a south easterly direction and with the conveyors for both the NWI and Roy Hill developments supported on pylons north east of the RHIO stockyard.

The proposed railway loops for NWI and Roy Hill impede the passage of storm surge for the 1 in 100 AEP storm surge events, such that there are widespread areas of reductions in peak storm surge levels to the south and east of the two proposed rail loops (as shown in Figure 4-14). The only areas where flood levels for the 1in 100 AEP event are projected to increase with the proposed NWI and Roy Hill developments are from the proposed culvert of the adopted railway spur line across South West Creek, for a distance of approximately 7 km downstream. No existing infrastructure would be affected by the increase in flood levels for the 1 in 100AEP event that would result from the proposed developments. Figure 4-13 shows that with the proposed NWI and Roy Hill developments, the towns of Wedgefield and South Hedland remain above the 1 in 100 AEP flood event.

Figure 4-15 shows that flow velocities for the 1 in 100 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek. For the 1 in 100 event with the proposed developments, the peak velocities reach 2.4 m/s near to the eastern end of the proposed Roy Hill railway loop. Roy Hill propose to install a revetment wall in this location to withstand velocities and flows from flooding and storm surge events. Peak velocities in the 1 in 100 AEP event near to the western edge of the existing FMG railway loop reach 1.2 m/s, which is the same as velocities at this location for the 1 in 100 event under existing conditions. Velocities in the 1 in 100 AEP event under with the proposed developments would exceed 1 m/s for a distance of approximately 1 km downstream of the proposed culvert crossing of South West Creek, with a maximum velocity of 2.0 m/s immediately downstream of the culvert. Scour protection may be required, particularly in the vicinity of the waterway openings in the conveyor and the culvert through the adopted railway spur line, to control erosion during flood and storm surge events.

Changes to Rainfall Runoff Rates

Installation of the a wharf, overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the northern portion (approximately 5km) of the western rail spur will cause increases in runoff rates from those that would be observed under the natural catchment conditions, due to increases in the impervious area. This represents 0.03% of the total combined catchment area of South and South West Creeks. This change in the volume of runoff produced is insignificant when compared with the overall total volume of runoff generated from the existing catchments of South and South West Creeks during flood events.

During minor rainfall events there will be additional runoff generated from these impervious areas that would otherwise runoff into areas surrounding the project site. Stormwater management practices, such as detention of runoff from impervious areas to reduce flow velocities and the prospects of scour, may be required to manage these local increases in runoff rates.

Impacts on Surface Water Quality

Rainfall on iron ore stockpiles and impervious surfaces around the rail loop and conveyor will cause runoff that may contain sediment and low levels of other contaminants. Silt trap and sedimentation basin facilities will be developed to trap sediments washed off the stockpile areas during rainfall events to prevent these materials flowing unmitigated into the receiving environment.

Modelling results indicate the velocities of flows in the floodplain around the site may change as a result of the development, although due to the relatively low level of encroachment of the project site into the floodplain these changes in velocity are expected to be small. Suspended sediment loads of downstream water may be increased if alteration of flow regimes results in scour velocities created downstream of the development. Areas where high flow velocities may be expected should be identified and appropriate measures taken to slow flow and contain sediments on site.

Management Measures

During rainfall events, some additional stormwater runoff will be generated from new impervious areas of the stockpiles, road train ring road, the twin cell rotary car dumper and the ancillary infrastructure buildings. On-site stormwater management measures may need to be implemented to detain the runoff produced from these areas and to minimise scour caused by direct runoff from these areas.

The stockpiles, car dumper and conveyor loader will be located above the 1 in 100 AEP flood and storm surge level, to minimise the probability of this infrastructure being inundated. In the event of a large flood or storm surge event that floods the rotary car dumper and the facilities for detaining water released during unloading of the rail cars, some sediment and other contaminants may be released into the environment. During a large flood event (rarer than 1 in 100 AEP), there would be large volumes of water inundating the flood plains of South and South West Creek so that there would be considerable dilution of the potentially contaminated water. Volumes of flood water would be sufficiently large that there would not be expected to be appreciable increases in the concentrations of contaminants.

Culverts will be designed to allow flood flows to pass through the at the South West Creek crossing during the 1 in 100 AEP event without causing inundation of the railway line. A regular programme of inspection and maintenance of the culverts would be implemented so that they continue to perform their function in passing flows during flood events. Waterway openings, in the form of bridges or culverts, will be designed in the conveyor embankment to pass the 1 in 100 AEP flood and storm surge event.

Baseline and continuous monitoring of sediment and other pollutants during construction and operation of the facility will be conducted to detect any changes in water quality due to these activities.

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Revision C	8 June 2011	M. Fieldhouse	M. Fieldhouse	8 June 2011	Minor revisions in response to comments from Coffey Environments and NWI

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1 Introduction

1.1 Background

North West Infrastructure (NWI) has proposed a development of a multi-user iron ore export facility for Port Headland in Western Australia. The proposed *Landside* project includes a two berth wharf, overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the northern portion (approximately 5km) of the western rail spur. The *Rail* project involves a rail connection between the western rail spur and conections to any combination of rail lines operated or proposed by FMG, BHP-BIO and RHIO rail lines as indicated in Figure 1-1.

Coffey Environments are compiling the Environmental and Social Impact Assessment (ESIA) referral document for the project. SKM were engaged by Coffey Environments on behalf of NWI to undertake the surface water hydrology input to the ESIA.

The proposed NWI rail loop, conveyor and port infrastructure is located immediately to the west of a similar railway loop, railway line, iron ore stockpile, conveyor and wharf facility previously referrd to the Environmental Portection Authority (EPA) (Roy Hill Infrastructure Pty Ltd, 2010) and subsequently approved in Bulletin 1380 (EPA 2011), as shown in Figure 1-1.

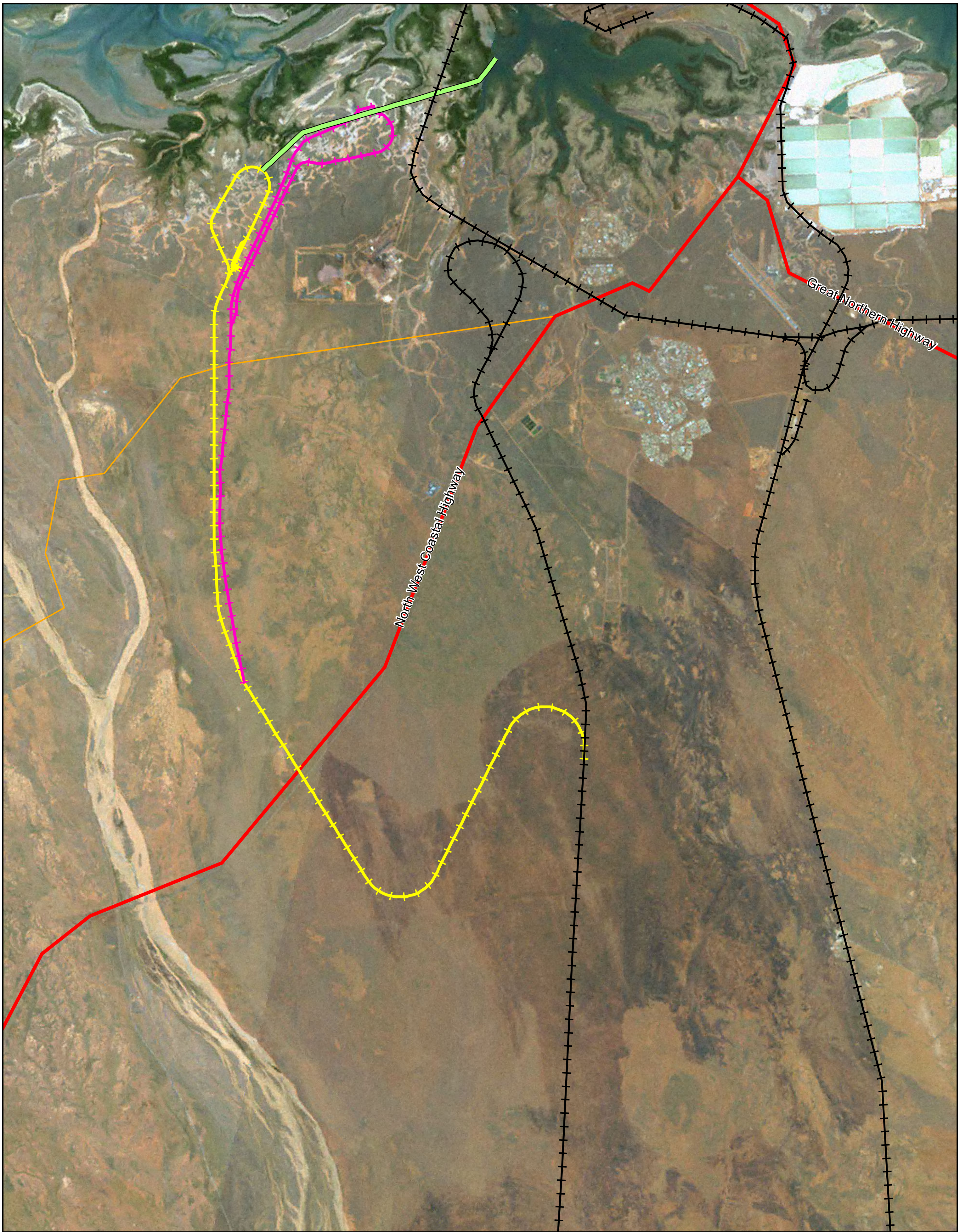
1.2 Scope of This Report

This report contains an assessment of the potential surface water impacts of the project and a discussion of mitigation measures to control those potential impacts. The terrain in the vicinity of the project site generally slopes from south to north, transitioning from broad, gently sloping plains to a flat intertidal zone to the north of the proposed NWI and Roy Hill railway loops.

Floods and storm surges are known to occur in the vicinity of the project site. Such events may result in inundation of some of the proposed infrastructure and modifying the direction, depth and velocity of flows in areas around the project site. As shown in Figure 1-1, the adopted railway connection from the FMG line crosses the drainage line of South West Creek. The NWI and Roy Hill lines split after crossing South West Creek and travel parallel to one another and in so doing also run parallel to the major drainage lines of South West Creek, South Creek and the Turner River.

This report uses a combined hydrological and hydraulic model to investigate flooding under existing conditions and to assess impacts of the proposed development and also the combined impact of the NWI and Roy Hill developments.

The proposed rail loop and conveyor are located on land with low elevation, often in the order of 1 to 2 m AHD. As a result, the proposed development will be subject to the influences of tidal cycles and storm tide effects. In addition to these, freshwater flooding from rainfall events do pass through the site, however generally these catchments are small and it is anticipated that the measures required to accommodate storm tide flows will comfortably accommodate the freshwater flows of equivalent recurrence interval. The adopted rail line from the rail loop across to the FMG railway line will also cross South West Creek, modifying flood flows from the catchment of South West Creek. The proposed Roy Hill Iron Ore infrastructure will have a combined influence on flooding in the area and the combined effects of the two proposed projects are assessed in this report.



Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 1-1 Locaility Plan

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1.3 Reliance Statement

The sole purpose of this report and the associated services performed by Sinclair Knight Merz Pty Ltd (SKM) is to assess the impacts on surface water flows and water quality for the proposed overland conveyor corridor, stockyard, road receival hopper, rail car dumper, rail loop, access roads and administration facilities, together with the western rail spur for the Multi-User Iron Ore Export Facility in Port Hedland in accordance with the scope of services set out in the contract between SKM and Coffey Enviornments. That scope of services, as described in this report, was developed Coffey Environments

In preparing this report, SKM has relied upon, and presumed accurate, certain information (or absence thereof) provided by the Client and other sources. Except as otherwise stated in the report, SKM has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

SKM derived the data in this report from a variety of sources. The sources are identified at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. SKM has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose of the project and by reference to applicable standards, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report.

This report should be read in full and no excerpts are to be taken as representative of the findings. No responsibility is accepted by SKM for use of any part of this report in any other context. Whilst high resolution and accurate LIDAR derived terrain data was used for hydraulic modelling across much of the study area, one part of the area modelled distant from the proposed rail loop was modelled using lower resolution terrain data derived from Landgate 10 metre contour interval data. Whilst this has insignificant impact on the depths and velocities in the vicinity of the proposed Project, depths and velocities derived in and close to the area of lower resolution terrain data will have a lower level of accuracy.

This report has been prepared on behalf of, and for the exclusive use of, Coffey Environments and NWI, and is subject to, and issued in connection with, the provisions of the agreement between SKM and Coffey Environments and NWI. SKM accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party.

2 Source Data Analysis

2.1 Climate Data

2.1.1 Rainfall Data

Monthly rainfall statistics for Port Hedland Airport were obtained from the Bureau of Meteorology's Climate Data On-line service. Table 2 shows the monthly rainfall statistics for this site for the entire period of climate record (1942-2011). Port Hedland is a generally arid area, with mean annual rainfall of 310 mm/year. The statistics show that there is a pronounced wet season, running from mid-December through to about June, with mean monthly rainfall totals exceeding 10 mm/month. The maximum monthly rainfall totals and maximum daily rainfall totals are observed in the months between December and April, which aligns with the period when Tropical Cyclones typically occur within the region. The period from July through November is consistently dry, with low rainfall totals, although thunderstorms can occur at any time of the year. Year to year variability and the general aridity of the area can result in entire months with zero recorded rainfall, even during the period of the year that is typically classified as the wet season.

- **Table 2 Monthly rainfall statistics for Port Hedland airport (derived from entire period of record 1942-2011)**

Statistic	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Mean monthly rainfall (mm)	59.4	93.2	48.7	23.0	27.2	21.2	11.1	5.0	1.3	0.9	2.5	18.4	310.9
Median monthly rainfall (mm)	21.4	74.6	15.7	2.3	8.3	6.8	2.7	0.6	0.4	0.2	0.0	0.5	307.1
Maximum monthly rainfall (mm)	453.5	360.0	427.2	352.1	169.9	128.6	80.5	58.6	27.4	8.2	66.8	219.0	626.8
Minimum monthly rainfall (mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	44.5
Highest recorded daily total in month (mm)	387.1	328.9	156.8	117.2	156.2	127.6	73.2	34.6	19.0	7.4	59.4	169.3	387.1
Mean number of days of rain	5.0	7.1	4.4	2.0	3.1	2.9	2.2	1.2	1.0	0.8	0.6	1.9	32.2

Design values adopted for flood modelling were from databases of design rainfall that are accepted industry standard for flood modelling, Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) and Cooperative Research Centre Focussed Rainfall Growth Estimation method for Western Australia (Durrant and Bowman, 2004).

2.1.2 Tidal and Storm Surge Data

The Indian Ocean in the Port Hedland area undergoes significant tidal variation. The Highest Astronomical Tide defines the tide level produced by the combined effects of the solar and lunar cycles only, with no allowance for storm surge conditions. Under these conditions, the peak water level in Port Hedland reaches 4.0 m AHD. The solar and lunar cycles are such that the Highest Astronomical Tide only occurs every 18.6 years.

Spring tides occur approximately every two weeks (at the time of full and new moons). The Mean of High Water Spring (MHWS) tides, estimated using the astronomical components only, is 2.83 m AHD at Port Hedland. For modelling the combined effects of relatively common flood events (for example the 1 in 10 annual exceedance probability (AEP)[1] event), using a downstream boundary condition of MHWS at 2.83 m AHD provides a reasonable estimate of the combined peak water levels of an event with an overall annual exceedance probability (AEP) of 1 in 10.

In addition to tidal effects, storm surges caused by Tropical Cyclones and other tropical storms will elevate water levels in the ocean above the tidally induced water level range. Global Environmental Modelling Systems (2000) includes a summary table of observed flood events in the Port Hedland region between 1917 and 2000. The peak storm surge level recorded in Port Hedland of 5.7 m in January 1939 caused inundation of the Hotel in Port Hedland (Jacka, 1994).

Global Environmental Modelling Systems (2000) used a hydrodynamic modelling approach to estimate storm surge levels in Port Hedland with AEP of 1 in 50 and 1 in 100. Global Environmental Modelling Systems (2000) identifies that the peak storm surge levels for at the Ocean in the vicinity of Port Hedland are 5.5 and 7.4 m AHD respectively.

2.2 Topography

The primary source of topographic data for the hydraulic model was a digital terrain model (DTM) derived using light detection and ranging (LIDAR), which covered the alignment of the proposed rail loop and connection to the FMG rail line. The LIDAR terrain data coverage has a horizontal resolution of 10 metres and a vertical accuracy (to one standard deviation) of +/- 100 mm.

Due to limitations in the extent of LIDAR data coverage for the DTM, levels for one part of the terrain were derived from lower resolution contour data from Landgate Western Australia. This dataset was derived from contour data with a contour interval of 10 metres and that was originally mapped on topographic maps with a scale of 1:50,000.

[1] The probabilty that a given flood peak or rainfall total accumulated over a given duration will be exceeded in any one year. Bureau of Meteorology explain the relationship between the AEP and ARI at http://www.bom.gov.au/hydro/has/ari_aep.shtml .

For the purposes of hydraulic modelling the two data sets were merged into a single DTM. Using the reasonable assumption that the LIDAR data is considerably more accurate than the Landgate contour data, this process of merging required some smoothing of the Landgate derived data near the intersection with the LIDAR data to provide for a sufficiently smooth transition in the terrain. Smoothing of the terrain in this way minimised artefacts in the hydraulic modelling results that could have resulted from sharp transitions in the DTM near the boundaries of the two data sets.

Catchments and subcatchments for the hydrological models of South and South West Creeks were derived using a combination of the Landgate 10 metre contour data (for the upper catchment) and the combined DTM described above that was also used for the hydraulic modelling. This data resolution provides sufficient resolution to define the drainage for each subcatchments in the hydrological model to sufficient accuracy.

The DEM data finished at the coastline. The coastline was therefore defined as a water level elevation boundary in the hydraulic model with water levels forced according to tidal and storm surge levels in the ocean.

2.3 Vegetation, Landuse and Landform

Vegetation across the catchments of South and South West Creek mainly consists of scattered shrub and grasslands. There are mangrove communities along the coastline in and near the inter-tidal zone.

The Project site is located across several pastoral leases and has been subject to cattle grazing in the past. Aerial photography and satellite imagery of the project area, along with a field survey, was used to assess catchment roughness values that were adopted in the hydraulic modelling.

2.4 Adopted Railway Spur Line, Rail Loop and Conveyor Alignment

Figure 4-5 shows the adopted railway spur line, rail loop and conveyor alignment associated with the proposed project. The adopted railway spur line, railway loop and conveyor will all be built up to above the 1 in 100 AEP flood level. Iron ore stockpiles will be located within the lower area in the centre of the railway loop. Drainage from the centre of the railway loop will be designed so that rainfall occurring within the rail loop can drain out but flap gates will be installed to prevent storm surge, floods and tides running into the centre of the rail loop where the iron ore stockpiles will be located.

3 Flood Hydrology

3.1 Catchment Description

The Project Area is located within the catchments of South and South West Creeks on the Pilbara Coast near Port Hedland. The Project area is in the Western part of this area and part of the proposed NWI adopted railway alignment runs along the divide between the catchments of South West Creek and the Turner River.

South and South West Creeks drain into Port Hedland Harbour, which then drain into the Indian Ocean. The catchments of South and South West Creeks are sufficiently flat (particularly in the northern part of each catchment) that during periods of flood or high tides and storm surge the flows from the two creeks combine into a single drainage system. During flood periods flows from the catchment of South Creek can cross over and flow into South West Creek and vice versa. The combined catchment area of South and South West Creeks is 557 km^2.

The Turner River is a much larger catchment than the combined catchments of South and South West Creeks, having a total catchment area of 3556 km^2. The alignment of the proposed Project is such that no surface water flows are expected from the Project into the Turner River and that flows in the Turner River would not be modified by the Proposed Project.

3.2 Hydrological Analysis Method

The hydrology of the project site was analysed using the RORB rainfall runoff routing model (Laurenson and Mein, 1992). The combined catchment of South and South West Creeks was subdivided into eighteen separate subcatchments, with subcatchments divided using the combined DTM from the Landgate 10 metre contour data and the LIDAR derived DTM for the proposed Project Area.

The RORB model was run for design rainfall events with AEP of 1 in 10, 50 and 100 and for rainfall event durations of 6, 12 and 24 hours. Since the level of urbanisation is low across the catchment, the entire catchment was modelled as pervious.

3.3 Catchment Design Rainfall Estimates

Design rainfall estimates for the combined catchment of South and South West Creek were estimated using established databases of design rainfalls for use in Western Australia. Design rainfall for 24 hour duration and 1 in 50 and 1 in 100 AEP events for catchment were determined from CRC FORGE database for WA (Durrant and Bowman, 2004) because this provides the most up to date and appropriate estimates for events in this AEP range. Areal reduction factors for all durations were determined from CRC FORGE work for WA (Durrant and Bowman, 2004), using the summer “rest of WA” equation.

Rainfall intensities for the catchment are derived for 24 hour duration events with AEP of 1 in 50 and 1 in 100 using the equation

CRC FORGE (Durrant and Bowman, 2004) only provides areal reduction factor (ARF)[2] and point design rainfall intensities for rainfall events with durations of 24 hours and longer. Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) provides ARF and point design rainfall intensities for shorter durations and AEP more common than 1 in 50 AEP. The estimates from Australian Rainfall and Runoff (1998) for design rainfall intensities and ARF have been effectively superseded by Durrant and Bowman (2004). However, the ratios of both ARF and design point rainfall intensity provide a useful means of extrapolating the design rainfall intensities for shorter durations from the estimates for 24 hours. Design intensities for the catchment for shorter durations were therefore determined using the following equation:

Table 3 lists the design rainfall depths for the AEP and the durations that were considered in hydrological modelling for the catchments of South and South West Creeks.

- **Table 3 Adopted design rainfall depths for design events on combined catchment area of South and South West Creeks**

Event Duration (hours)	Design Rainfall Depth in mm for Event with AEP			
	1 in 10	1 in 20	1 in 50	1 in 100
6	89	114	151	175
12	124	161	215	251
24	143	186	249	292

Temporal patterns for the design rainfall were derived from the patterns in Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) for Zone 7.

The spatial pattern of rainfall during the design events was assumed to be uniform across the 557 km^2 combined catchment area of South and South West Creeks.

[2] Ratio between the design rainfall intensity for a given burst duration and annual exceedance probability for a particular catchment area to the design rainfall intensity for the same duration and annual exceedance probability at a point.

3.4 Verification of RORB Model to Regional Flood Frequency Analysis

There are no streamflow gauging stations within the catchments of South or South West Creeks. It was therefore not possible to perform direct calibration of the RORB model to observed flood events within the catchment.

The parameters of the RORB model were verified to flood peak quantiles estimated from a regional frequency analysis of peak flows recorded from five gauging stations in the Fortescue River and Portland Coast Basins. Sites with recorded flow data were selected from those available in those basins, on the basis of sites that had at least 20 years of streamflow record and catchment areas between 40 and 1200 km^2. These gauging stations are as listed in Table 4.

- **Table 4 Streamflow Gauging Stations Used in Regional Flood Frequency Analysis**

Site Number	Site Name	Catchment Area (km²)	Period of Available Data	Years of Peak Flow Record Used in Analysis
709007	Harding River at Marmurrina Pool U-South	49	Aug. 1974 – May 1999	24
709006	Tanberry Creek at Blue Dog Pool	128	Mar. 1974 – May 2001	27
708227	Portland River at Recorder Pool	553	Nov. 1966 – May 2001	35
709010	Turner River at Pincunah	885	Jan. 1985 – Still open	18
709001	Harding River U/S Cooya Pooya	1058	Dec. 1965 – Jan. 1985	20

Regional flood frequency analysis was used to fit a combined flood frequency curve to the data from the five sites. L-Moment analysis (Hosking and Wallis, 1997) was used to fit a Generalised Extreme Value (GEV) distribution to the annual maxima for each of the sites. The fitting procedure used weighted averages of the higher order L-Moments so that the regional frequency curve has a consistent shape to the pooled data from all five sites. A regression equation was fitted between catchment area and the mean annual flood (first L-Moment value) derived for each of the five regional sites. The regression equation with catchment area was then used to determine the mean annual flood for the South and South West Creeks catchment, with the shape and parameters of the GEV distribution derived from the regional flood frequency analysis to derive peak flows for design floods for the combined catchment area of South and South West Creek. Peak flows for design events from the regional frequency analysis are provided in Table 5.

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- **Table 5 Verification of flood peaks produced by the RORB model at the combined outlet of South and South West Creeks to flood peaks estimated from regional flood frequency analysis**

Event	Growth Factor from Regional Frequency Analysis*	Regional Frequency Analysis Peak Flow Estimate (m^3/s)	RORB Model Peak Flow Estimate (m^3/s)
Mean Annual Flood	1	299	Not modelled
1 in 10 AEP	2.28	681	662
1 in 50 AEP	4.85	1450	1420
1 in 100 AEP	6.44	1925	1876

* Growth factor = Ratio of peak for a given AEP derived from the regional frequency analysis to the Mean Annual Flood

The RORB model was run for design floods with AEP of 1 in 10, 50 and 100. The value of the catchment delay parameter, k_c, for the RORB model was derived from the regional prediction equation contained in Book V of Australian Rainfall and Runoff (Institution of Engineers Australia, 1998) for the Northwest, Kimberley and Wheatbelt regions, providing a value of $k_c = 32$. In accordance with the regional equation in Book V of Australian Rainfall and Runoff, the catchment non-linearity parameter *m* was set to a value of 0.8. The initial and continuing loss parameter values were adjusted until a fit was obtained between the peak flows derived from the RORB model and the peak flows derived from the regional frequency analysis. This verification process resulted in adopting an initial loss of 37 mm and a continuing loss of 7 mm/h. Flood peaks estimated by the RORB model for the design events are within 3% or less of peak flows from the regional flood frequency analysis, as shown in Table 5. While the continuing loss parameter value in particular is high, it is consistent with the loss rate that would be expected from a catchment in the arid Pilbara region. For all AEP that were modelled using RORB, the critical duration was 12 hours.

3.5 Hydrographs for Design Floods

The RORB model was run with the adopted parameters produced from the verification to the regional flood frequency analysis. Design flood hydrographs were extracted for the event with the critical duration that matched the peak flows from the regional flood frequency analysis. Separate flood hydrographs were extracted for each of the eighteen individual subcatchments in the RORB model. These were then used as direct inputs to the MIKE FLOOD hydraulic model.

4 Flooding Characteristics of the Project Area

4.1 Overview of Approach

Hydraulic modelling was used to assess the drainage characteristics of the Project site during several different flood scenarios. A model of the area containing the Project site was created using a MIKE FLOOD dynamically coupled one-dimensional and two-dimensional hydraulic model.

Some parts of the Project site footprint would be expected to be inundated by storm surge and high tide, rainfall derived runoff in the catchments of South and South West Creeks or by a combination of storm surge and rainfall derived runoff. A Tropical Cyclone approaching or crossing the coast in the vicinity of Port Hedland is likely to lead to a combined storm surge and rainfall runoff flooding event in this area. The response time of the catchments of South and South West Creeks are sufficiently short (approximately 12 hours critical duration) that it is likely that the storm surge and tidal flooding will peak at around the same time as the peak of rainfall runoff generated flooding in South and South West Creeks. The flood modelling scenarios adopted therefore consider a coincident storm surge and creek flooding event. For a given AEP, this approach is likely to produce conservative estimates of peak flood levels because there is some likelihood that the peaks of storm surge and river flooding events will not coincide. The scenarios modelled to estimate peak flood levels were as shown in Table 6.

- **Table 6 Combined catchment runoff and storm surge events modelled**

Nominal Overall AEP of Event	AEP of Creek Runoff Flood Event	Tidal and Storm Surge Boundary Condition
1 in 10 AEP	1 in 10 AEP	Mean High Water Spring Tide Level (constant head boundary)
1 in 50 AEP	1 in 50 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 50 AEP storm surge event, sinusoidally varying temporal pattern
1 in 100 AEP	1 in 100 AEP with flood peak at catchment outlet coincident timing to storm surge peak	1 in 100 AEP storm surge event, sinusoidally varying temporal pattern

The developed state was modelled by introducing the proposed NWI railway loop, conveyor embankment and adopted railway spur line into the model. The developed state model also includes the proposed impact of the Roy Hill loop, conveyor embankment and adopted railway spur line, so that the combined effects of both developments could be assessed. The terrain layer of the digital terrain model was modified by increasing the terrain height to well above the projected 1 in 100 AEP flood level, so that the railway line will not be inundated by such an event. Openings were then introduced into the model to represent culvert or bridge structures to pass flows through the adopted railway spur line and underneath the conveyor. Finally, the terrain was modified by introducing training embankments that will direct flows from South West Creek through the waterway at the creek crossing of the spur line instead of allowing those flows to continue along the upstream side of the adopted railway spur line.

4.2 Base Case Flood Modelling

4.2.1 Model Development

Hydraulic modelling was used to evaluate the flooding characteristics of the Project site in its existing state and assess the potential impact of flood flows on the proposed plant site and access road. SKM used MIKE FLOOD, a dynamically coupled MIKE 21 two-dimensional and MIKE 11 one-dimensional hydrodynamic model developed by DHI, to give an accurate simulation of the hydraulic behaviour of the Project site.

4.2.2 Terrain data Inputs

MIKE 21 is a two-dimensional hydraulic model, which represents the broader topography of the Project area in the form of a model grid. As discussed in Section 2.2, terrain data for the hydraulic modelling domain was derived mainly from LIDAR data captured for the Project, with additional data for missing regions on the outskirts of the project area derived from Landgate 10 metre contours. The horizontal grid resolution of the DTM for the hydraulic model was 10 metres.

4.2.3 Runoff Hydrograph Inputs

Rainfall runoff hydrographs from South and South West Creeks upstream of the Project site and from the Local sub-catchments were input into the model grid in a number of different locations to represent the distribution of rainfall runoff over the project site.

4.2.4 Tidal and Storm Surge Boundary Condition

The Indian Ocean in the Port Hedland area undergoes significant tidal variation. The Highest Astronomical Tide defines the tide level produced by the combined effects of the solar and lunar cycles only, with no allowance for storm surge conditions. Under these conditions, the peak water level in Port Hedland reaches 3.6 m AHD. The solar and lunar cycles are such that the Highest Astronomical Tide only occurs every 18.6 years.

Spring tides occur approximately every two weeks (at the time of full and new moons). The Mean of High Water Spring (MHWS) tides, estimated using the astronomical components only, is 2.83 m AHD at Port Hedland. For modelling the combined effects of relatively common flood events (for example the 1 in 10 AEP event), using a downstream boundary condition of MHWS at 2.83 m AHD provides a reasonable estimate of the combined peak water levels of an event with an overall AEP of 1 in 10.

In addition to tidal effects, storm surges caused by Tropical Cyclones and other tropical storms will elevate water levels in the ocean above the tidally induced water level range. Global Environmental Modelling Systems (2000) includes a summary table of observed flood events in the Port Hedland region between 1917 and 2000. The peak storm surge level recorded in Port Hedland of 5.7 m in January 1939 caused inundation of the Hotel in Port Hedland.

Global Environmental Modelling Systems (2000) used a hydrodynamic modelling approach to estimate storm surge levels in Port Hedland with AEP of 1 in 50 and 1 in 100. The results of the Global Environmental Modelling Systems (2000) hydrodynamic modelling of the storm surges were used to inform the boundary condition hydrographs at the Indian Ocean. Storm surge levels from Global Environmental Modelling Systems (2000) vary from east to west across the coastal boundary of the model domain. Sinusoidally varying storm surge and tidal boundary conditions were entered into the model with the following peak water levels for the 1 in 50 AEP and 1 in 100 AEP design flood and storm surge scenarios as shown in Table 7.

- **Table 7 Modelled Peak Storm Surge Level Hydrographs at Coastline of Hydraulic Model (from Global Environmental Modelling Systems, 2000)**

Model Scenario	Peak Storm Surge Level at Coastline (m AHD)		
	Western End of Model Domain	Middle of Model Domain (near proposed rail loop)	Eastern End of Model Domain (Port Hedland Harbour)
1 in 50 AEP Storm Surge	7.5	5.5	5.35
1 in 100 AEP Storm Surge	8.6	7.4	6.0

4.2.5 Roughness and Eddy Viscosity

Whilst there are local variations in vegetation cover across the study area, there is generally scattered shrub vegetation across most of the study area. It was therefore assumed that there would be consistent hydraulic roughness across the study area in the hydraulic model. A consistent Manning's n value of 0.032 was adopted across the entire study area. A consistent value of eddy viscosity of 2 m^2/s.

4.2.6 Representation of Railway Lines, Roads and Structures

There are a number of existing road and railway crossings of waterways within the study area. Nine of these waterway crossings were represented in the MIKE-FLOOD model using one dimensional hydraulic modelling structure elements. Dimensions for culverts and bridge water way openings for each of these structures were determined from field inspection undertaken of the study area in February 2011.

The impacts on flood flows of the proposed NWI conveyor and conveyor pylons from the rail loop to the wharf have not been explicitly represented in the hydraulic model. The conveyor will be designed to be above the 1 in 100 AEP storm surge level and will not impact upon flood flows. Pylons to support the conveyor will be sufficiently small in diameter and at sufficiently wide spacing to have minimal impact on overall flood levels upstream of the project site. The proposed Roy Hill conveyor will also be elevated and supported on pylons, with pylons for their conveyor spaced at intervals of approximately 50 metres (Worley Parsons, 2010). Flood levels in the local vicinity of the conveyor pylons (for both the NWI and Roy Hill conveyors) may be up to 50 mm higher than indicated by the overall flood modelling.

4.2.7 Assumptions

At the time of this assessment no recorded historic flood levels could be obtained for the Project site. This lack of data prevented the hydraulic model from being calibrated against recorded flood data. For the purposes of this assessment it is assumed that all rainfall runoff inputs and tidal data used in the modelling are accurate, and therefore the model is a sufficiently reliable representation of the hydraulic characteristics of the Project site and the floodplains of South and South West Creek.

The accuracy of results obtained from the hydraulic model is determined by the accuracy of the raw digital elevation data used. An assumption has been made during this assessment that the topography supplied for this assessment as well as the interpolation of that raw data into a topographic grid is an accurate representation of the existing topography of the Project site. This assumption was necessary as no ground truthing was able to be conducted at the Project site during this assessment.

The accuracy of model results is also dependent on the accuracy of runoff hydrographs input into the model and water output from the model at the downstream boundary. The has been no streamflow gauging in the catchments of South or South West Creeks, so parameter values for the hydrological model were verified to regional flood frequency analysis. This provides a degree of confidence in the runoff input into the model. The fixed sea level boundary used in all simulations has not been verified with observations or historic records of actual tidal ingress into the Project site. The assumption that the tide accesses the Project site to the extent estimated may be a conservative over estimate of tidal heights at the site.

The accuracy of flood levels and depths obtained from modelling results for this assessment is limited by these assumptions, which present the possibility of a compound error. This error is estimated to be relatively small due to the use of accurate topographic data (accuracy within 0.1 m), and the use of well established design event rainfall runoff estimation methods.

In terms of comparing the base case and developed case modelling results, the errors are the same, or in other words like is being compared with like. This results in a high level of accuracy when evaluating the changes to flood waters which may potentially result from the proposed development.

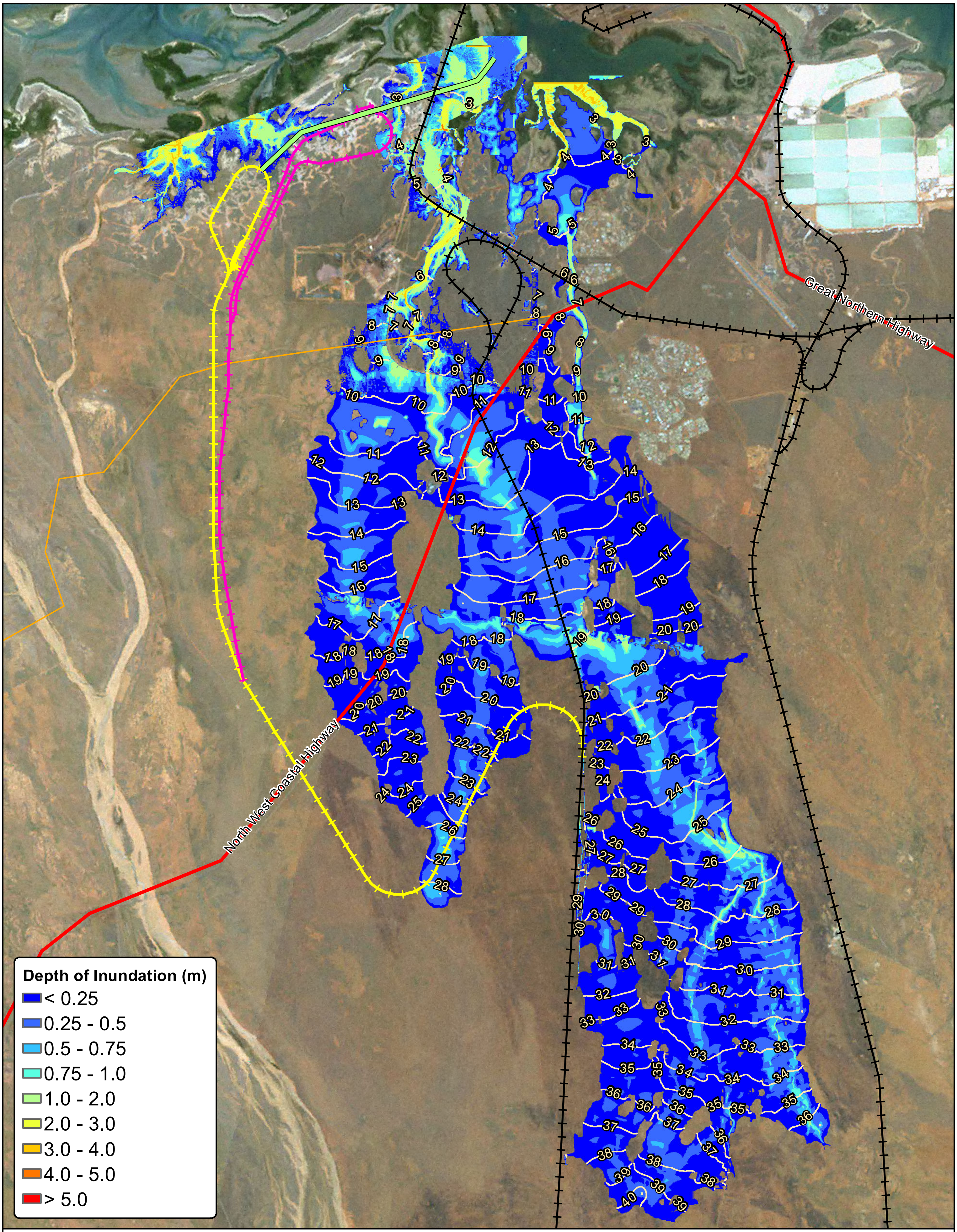
4.2.8 Base Case Modelling Results

The purpose of the base case assessment was to evaluate the flooding characteristics of the Project site in its existing state, prior to the commencement of the proposed Project. Three different rainfall design events and storm surge floods were simulated, with nominal AEP of 1 in 10, 50 and 100. The adoption of high downstream tailwater conditions in conjunction with flood inflows produces conservatively high inundation extents for each of the simulated events.

In the 1 in 10 AEP event under existing conditions, Figure 4-1 shows that the combined high tide and storm surge floods the intertidal zone to the north of the proposed NWI and Roy Hill railway loops. Flooding also occurs across a broad area of the floodplain of South West Creek, to the east of the existing FMG railway line, and a broad area of the floodplain of South Creek, between the FMG and BHP Billiton Iron Ore railway lines. Depths of flooding across the floodplains in the 1in 10 AEP event are less than 1 metre although there would be depths of flow of up to 3 metres in the centre of the channel of South West Creek. The towns of Wedgefield and South Hedland are above the 1 in 10 AEP flood level under existing conditions. Figure 4-2 shows that modelled velocities of flow for the 1 in 10 AEP event across the most of the floodplain areas of South and South West Creeks are less than 1 m/s.

Figure 4-3 shows that the 1 in 50 AEP storm surge under existing conditions would flood the entire area where NWI and Roy Hill are proposing to locate their railway loops and iron ore stockpiles. In the 1 in 50 AEP event under existing conditions, broad areas of the floodplains of South West and South Creeks would be inundated, although the depth of inundation is typically less than 1 metre, except in the middle of the drainage channels for the two creeks. The towns of Wedgefield and South Hedland are above the 1 in 50 AEP flood level under existing conditions. Figure 4-4 shows that modelled velocities of flow for the 1 in 50 AEP event across the most of the floodplain areas of South and South West Creeks are less than 1 m/s.

The 1 in 100 AEP storm surge for existing conditions (shown in Figure 4-5) demonstrates a similar pattern to the 1 in 50 AEP event, with the entire area proposed for the NWI and Roy Hill railway loops and iron ore stockpiles being flooded. The 1 in 100 AEP event results in flooding of the floodplains of South and South West Creeks, although the towns of Wedgefield and South Hedland are largely above the 1 in 100 AEP flood level. There is some flooding along minor drainage lines within South Hedland and around Wedgefield in the 1 in 100 AEP event, but it is not anticipated that there would be any over-floor flooding of buildings within either town under existing conditions. Figure 4-6 shows the pattern of velocities under existing conditions for the 1 in 100 AEP flood event, with velocities exceeding 1.0 m/s in central channels.



Legend

Peak Water Level (mAHD)

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-1 Existing Case 10 year ARI Peak Inundation and Peak Water Level

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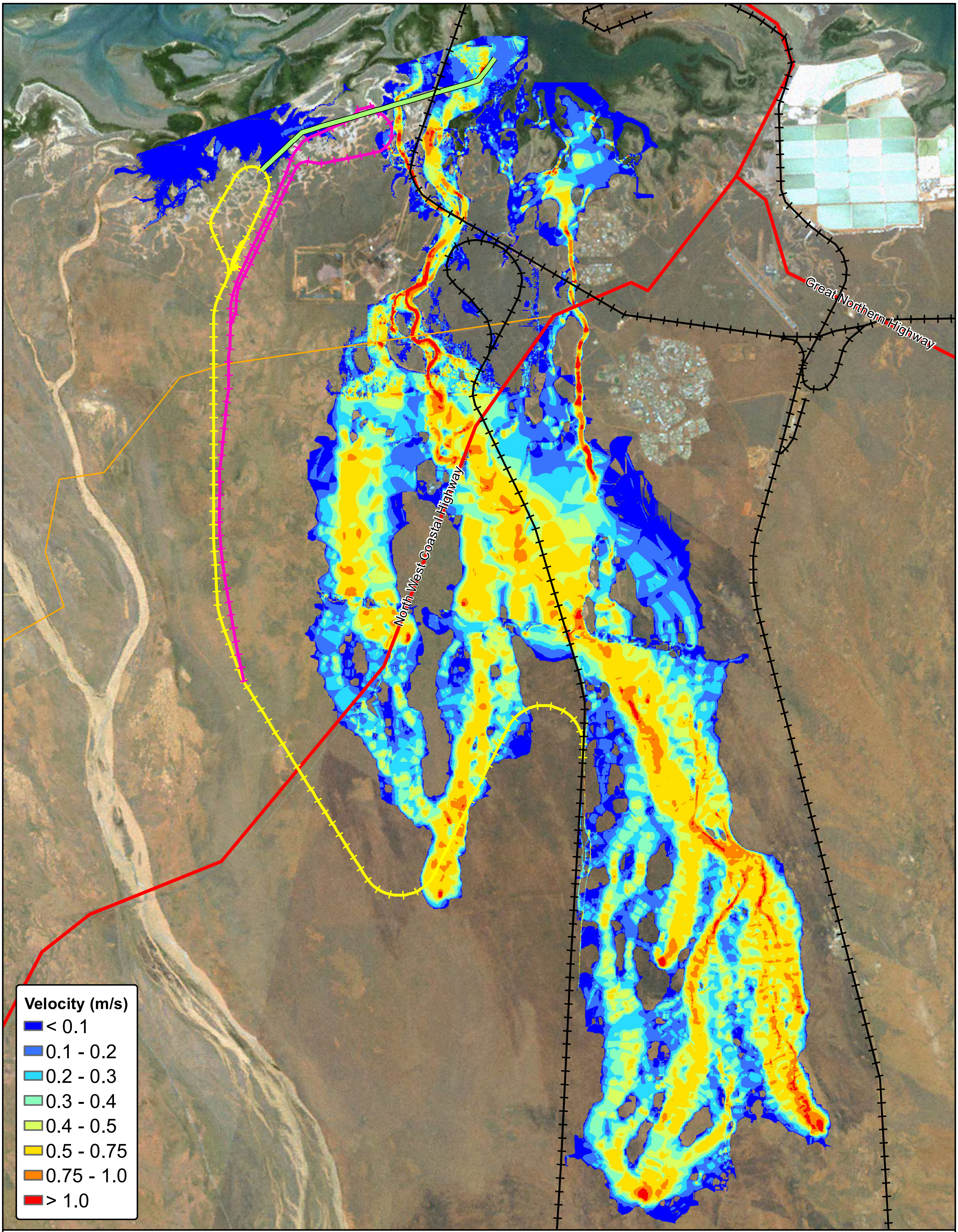

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-2 Existing Case 10 year ARI Peak Velocity

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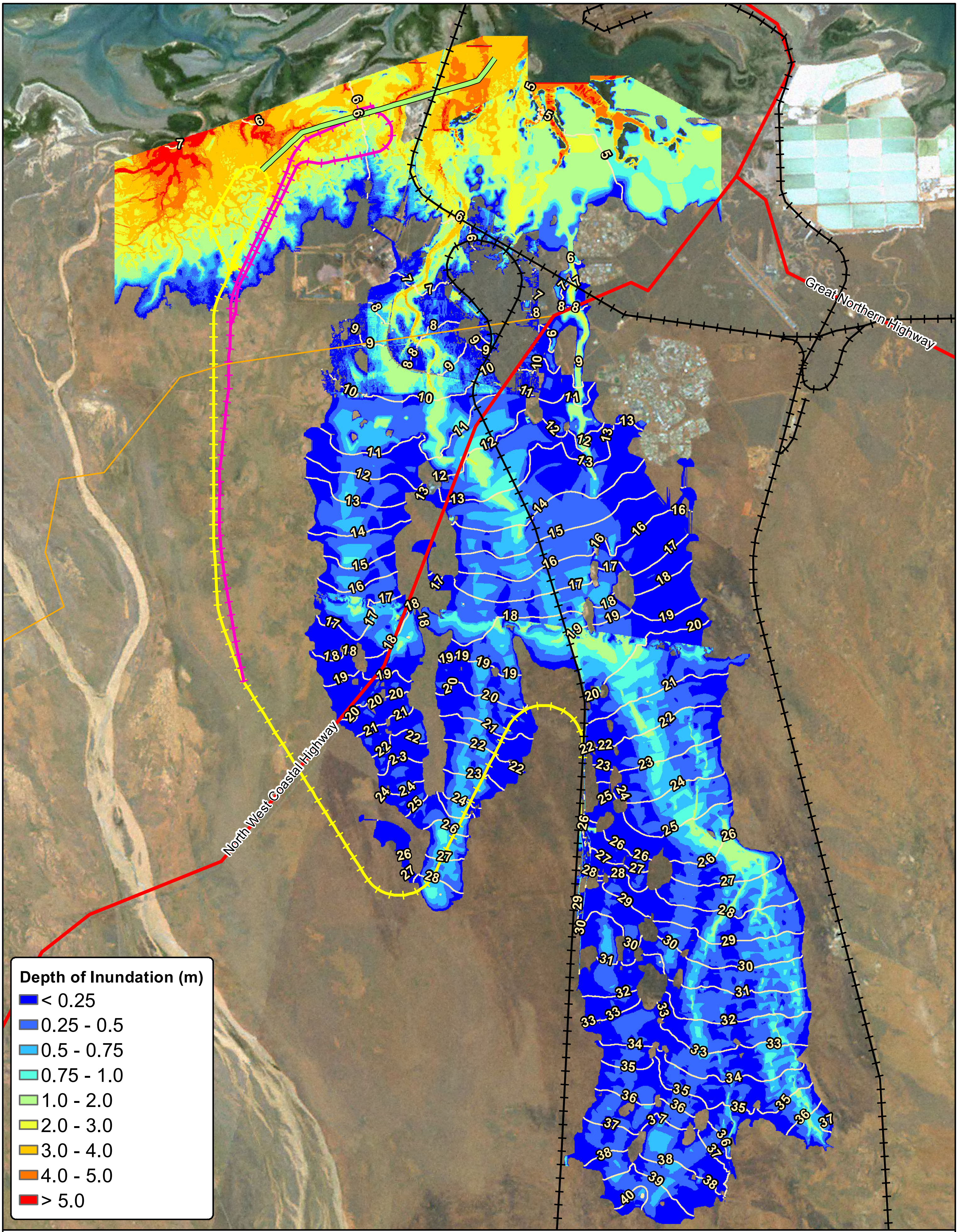

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-3 Existing Case 50 year ARI Peak Inundation and Peak Water Level

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By: S. Bayly

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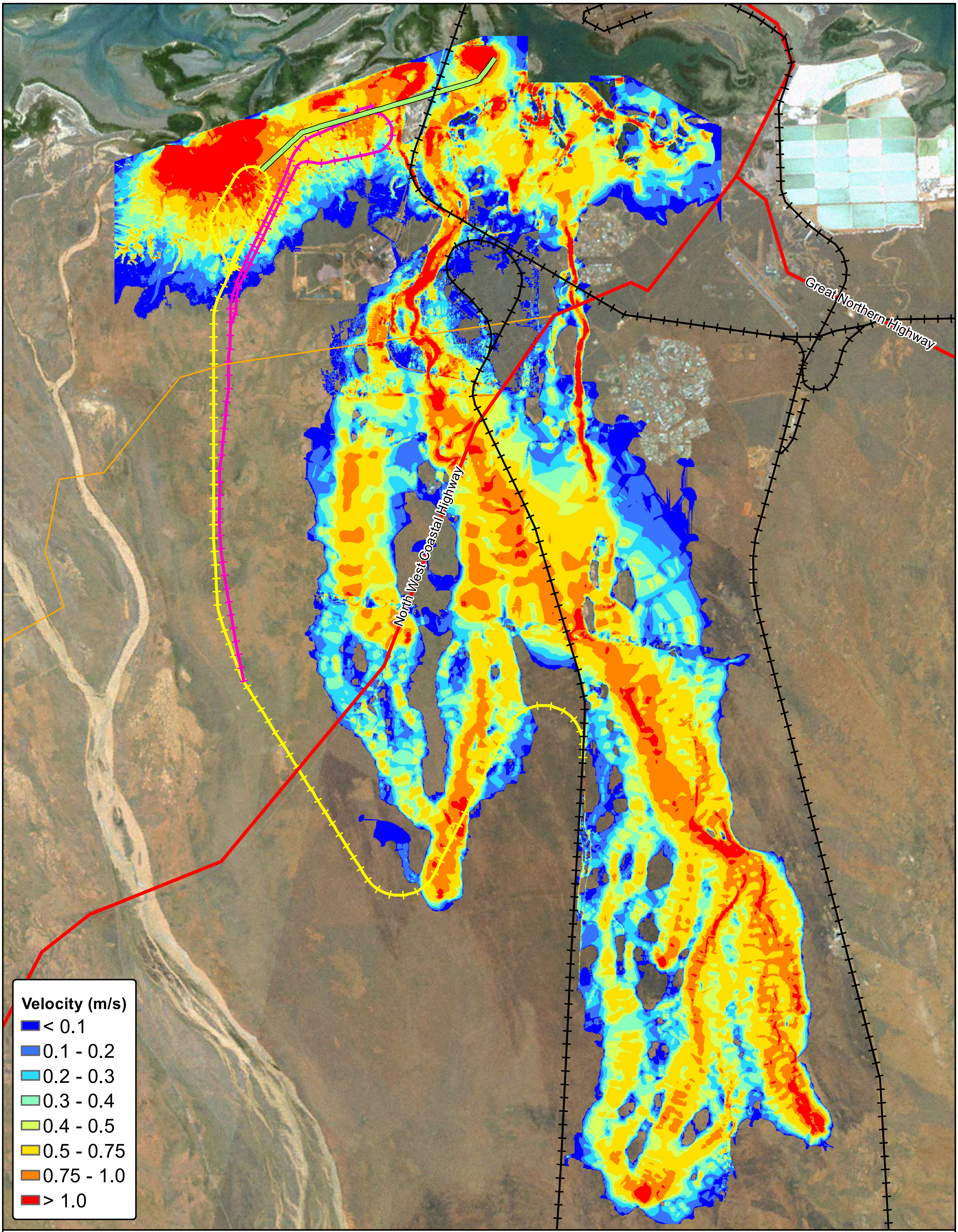

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-4 Existing Case 50 year ARI Peak Velocity

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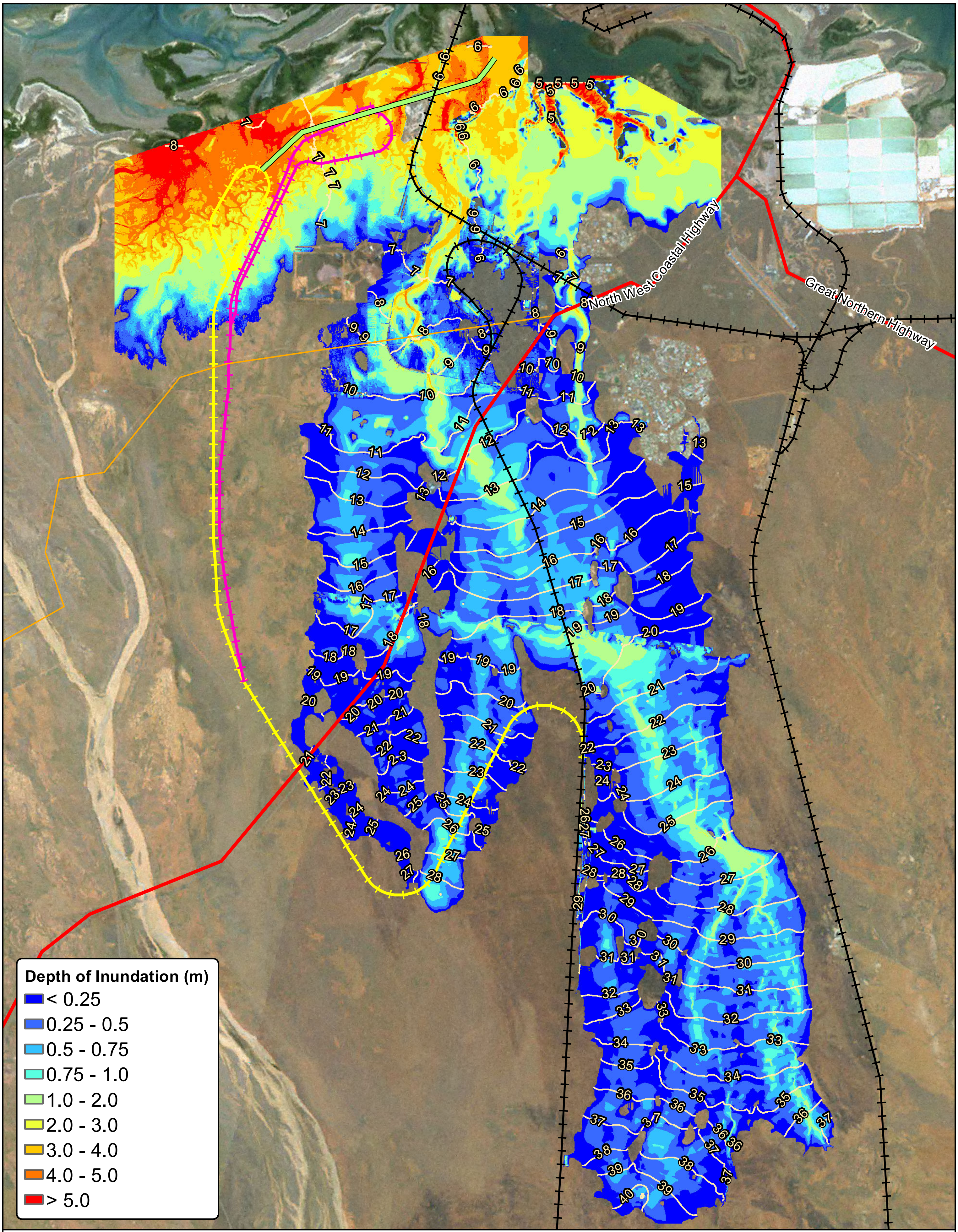

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Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-5 Existing Case 100 year ARI Peak Inundation and Peak Water Level

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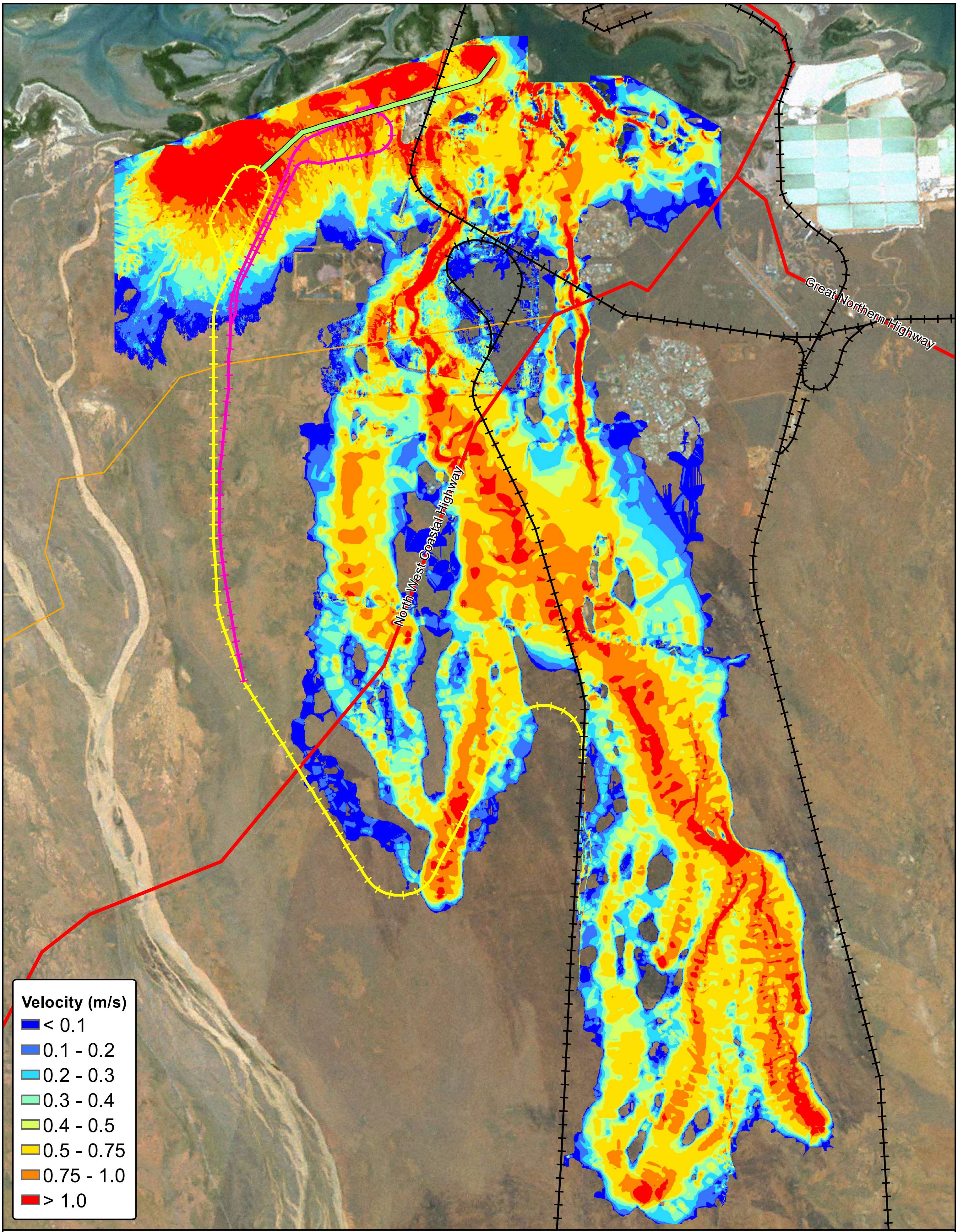

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-6 Existing Case 100 year ARI Peak Velocity

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4.3 Developed Case Flood Modelling

4.3.1 Model Development

The developed case model was derived from the model for the existing case, with changes to the terrain and structures to represent the railway loop and stockyard, adopted railway spur line and the conveyor embankment. The model terrain was modified to increase levels for each of these features in the proposed development case to well above the 1 in 100 AEP flood level. In addition, waterway openings were introduced into the adopted railway spur line and the conveyor embankment to allow for the passage of flood and storm surge flows. An embankment on the eastern side of South West Creek, upstream of the adopted railway spur line crossing of South West Creek, was introduced to direct flows through the culvert crossing by the spur line of South West Creek. In order to contain and direct the 1 in 100 AEP flood flows along South West Creek, the embankment was required to extend for 1.6 km in a south easterly direction away from the railway embankment to the north western side of the culvert.

4.3.2 Impact Assessment

The model was run for the 1 in 10, 1 in 50 and 1 in 100 AEP flood events including the proposed NWI and Roy Hill developments, with the proposed mitigation measures of the culverts where the proposed adopted railway spur line crosses South West Creek, the training embankment extending upstream of the South West Creek culvert crossing for 1.6 km in a south easterly direction and with the conveyors for both the NWI and Roy Hill developments supported on pylons.

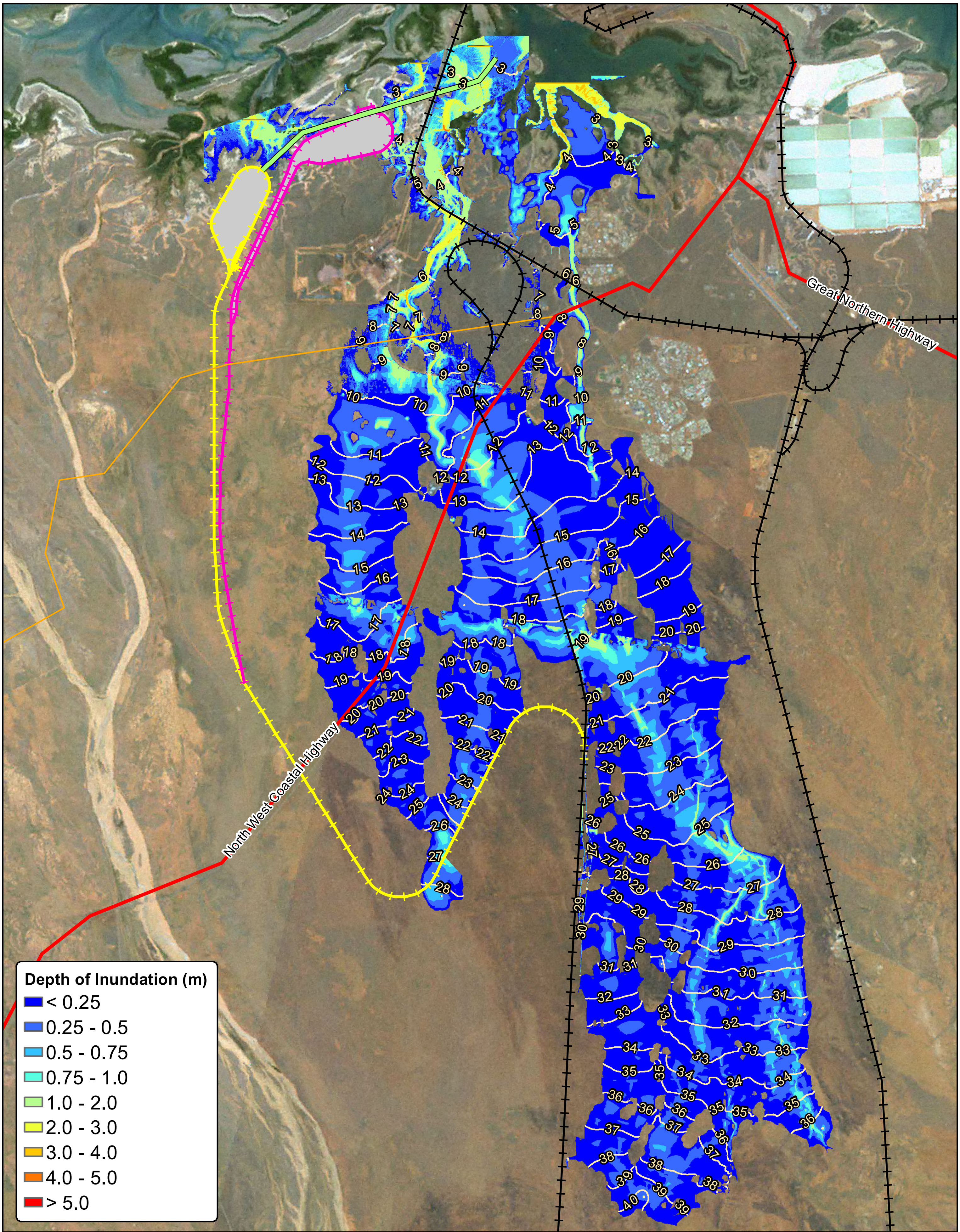
The pattern of flooding for the 1 in 10 AEP event with the proposed NWI and Roy Hill developments is very similar to the pattern of flooding under existing conditions, as can be seen by comparing Figure 4-7 to Figure 4-1. With the proposed NWI and Roy Hill developments, both Wedgefield and South Hedland remain unaffected by the 1 in 10 AEP flood event. The afflux map for the 1 in 10 AEP event (Figure 4-8) shows that flood levels are projected to increase by up to 0.3 m where the proposed railway connection line to the FMG line would cross South West Creek and that the projected increase in flood levels dissipates to the point that there is no afflux for all locations more than 5 km downstream of the culvert crossing. No existing infrastructure would be affected by the increase in flood levels for the 1 in 10 AEP event that would result from the proposed developments. Figure 4-9 shows that flow velocities for the 1 in 10 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek.

The proposed railway loops for NWI and Roy Hill impede the passage of storm surge for the 1 in 50 and 1 in 100 AEP storm surge events. As a result, the affluxes in the 1 in 50 AEP event (as shown in Figure 4-11) show reductions in peak flood levels of more than 0.15 metres across the area to the south of the proposed Roy Hill railway loop and to the west of the BHP Billiton railway line and reductions of peak flood levels of about 0.1 metres in the area to the east of the BHP

Billiton railway line. The only areas where flood levels are projected to increase with the proposed NWI and Roy Hill developments are from the proposed culvert of the adopted railway spur line across South West Creek, for a distance of approximately 7 km downstream. No existing infrastructure would be affected by the increase in flood levels for the 1 in 50 AEP event that would result from the proposed developments. Figure 4-10 shows that with the proposed NWI and Roy Hill developments, the towns of Wedgefield and South Hedland remain above the 1 in 50 AEP flood event. Figure 4-12 shows that flow velocities for the 1 in 50 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek.

The proposed railway loops for NWI and Roy Hill impede the passage of storm surge for the 1 in 100 AEP storm surge events, such that there are widespread areas of reductions in peak storm surge levels to the south and east of the two proposed rail loops (as shown in Figure 4-14). The only areas where flood levels for the 1 in 100 AEP event are projected to increase with the proposed NWI and Roy Hill developments are from the proposed culvert of the adopted railway spur line across South West Creek, for a distance of approximately 7 km downstream. No existing infrastructure would be affected by the increase in flood levels for the 1 in 100AEP event that would result from the proposed developments. Figure 4-13 shows that with the proposed NWI and Roy Hill developments, the towns of Wedgefield and South Hedland remain above the 1 in 100 AEP flood event.

Figure 4-15 shows that flow velocities for the 1 in 100 AEP event would also be virtually unchanged by the proposed developments from existing conditions, other than the area near the culvert crossing for the proposed NWI railway line of South West Creek. For the 1 in 100 event with the proposed developments, the peak velocities reach 2.4 m/s near to the eastern end of the proposed Roy Hill railway loop. Roy Hill propose to install a revetment wall in this location to withstand velocities and flows from flooding and storm surge events. Peak velocities in the 1 in 100 AEP event near to the western edge of the existing FMG railway loop reach 1.2 m/s, which is the same as velocities at this location for the 1 in 100 event under existing conditions. Velocities in the 1 in 100 AEP event under with the proposed developments would exceed 1 m/s for a distance of approximately 1 km downstream of the proposed culvert crossing of South West Creek, with a maximum velocity of 2.0 m/s immediately downstream of the culvert.



Legend

Peak Water Level (mAHD)

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-7 Developed Case 10 year ARI Peak Inundation and Peak Water Level

Job No: QE09828
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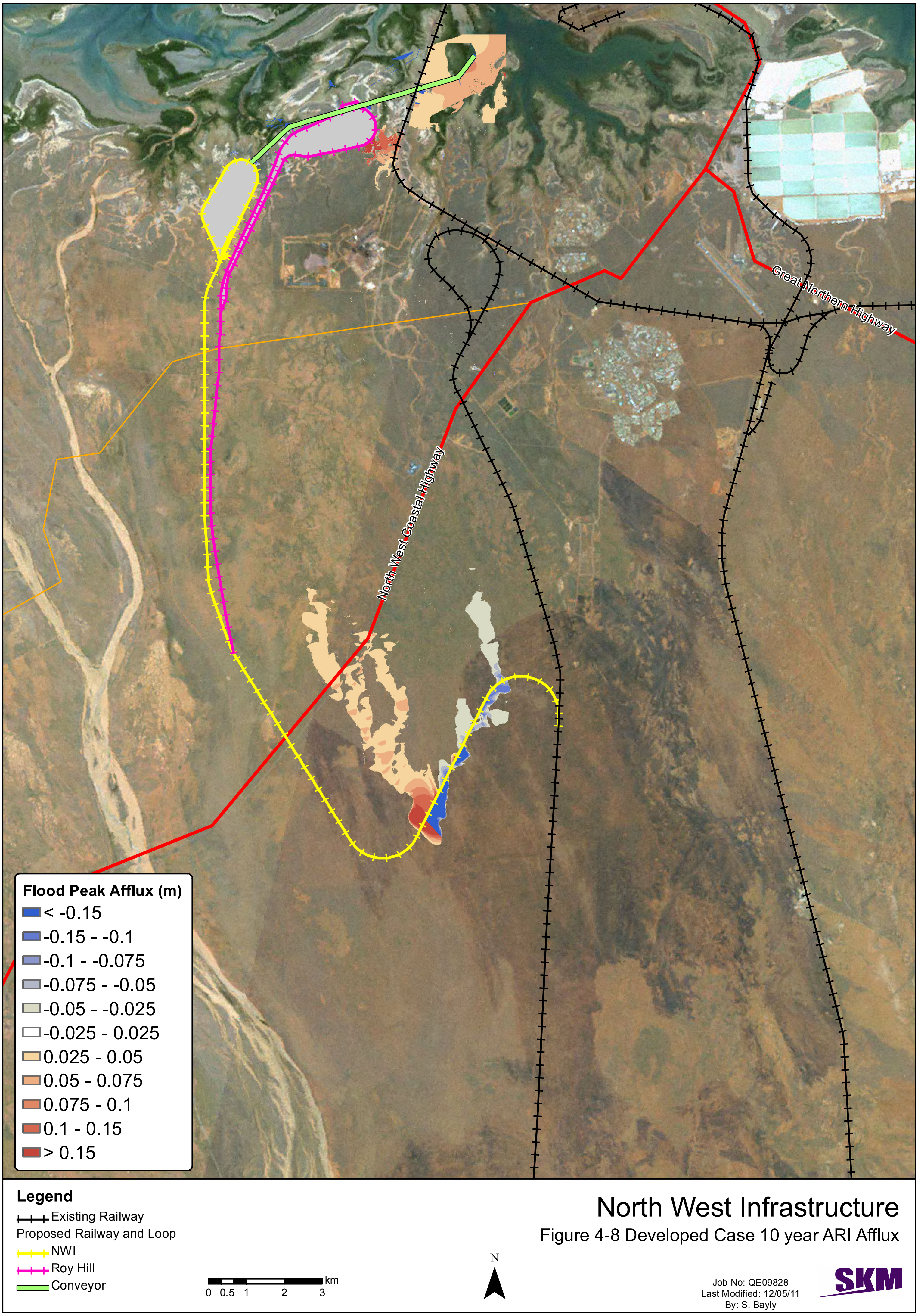
By: S. Bayly

SKM

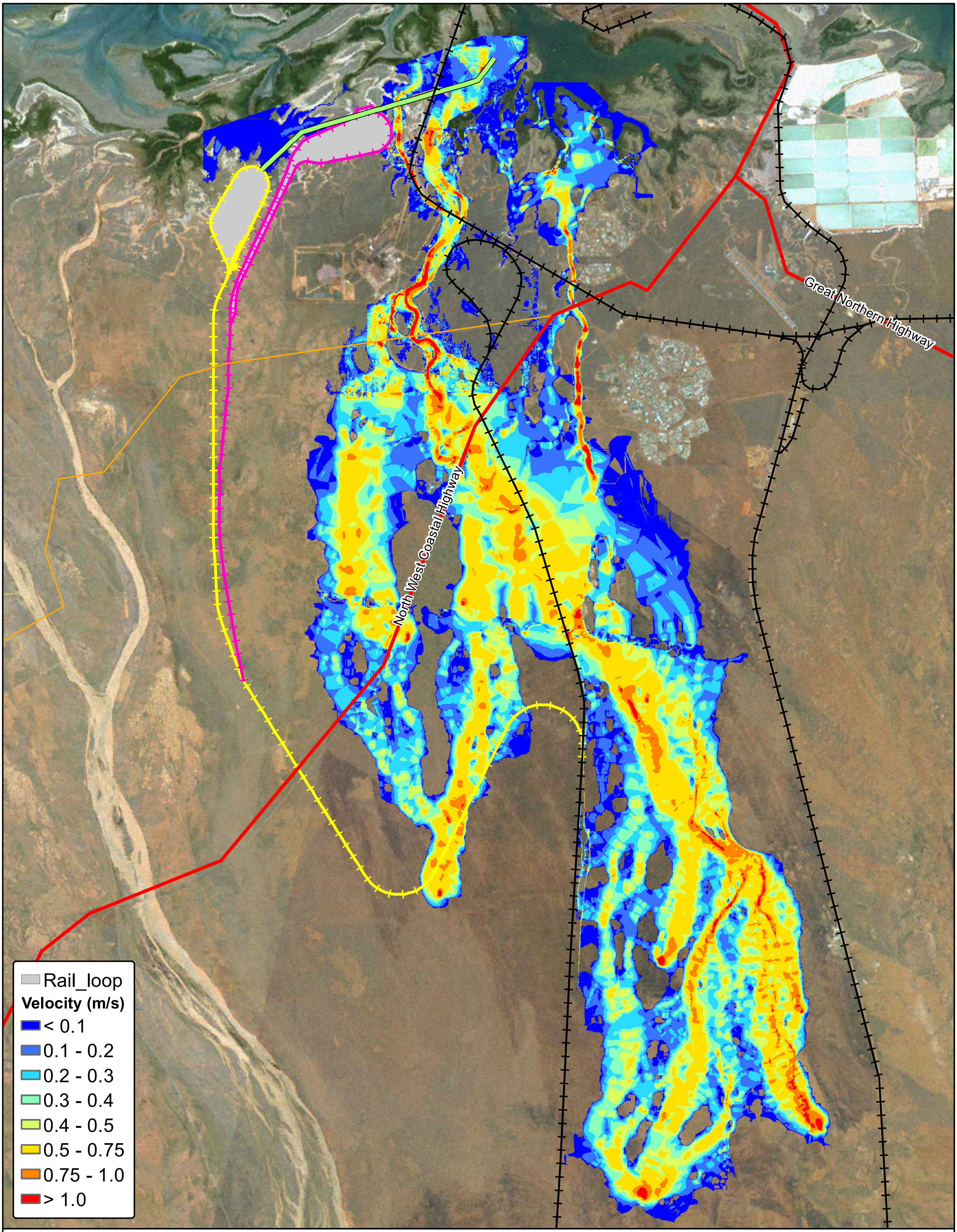


North West Infrastructure

Figure 4-8 Developed Case 10 year ARI Afflux

Job No: QE09828
Last Modified: 12/05/11
By: S. Bayly

SKM



Legend
Existing Railway
Proposed Railway and Loop
NWI
Roy Hill
Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-9 Developed Case 10 year ARI Peak Velocity

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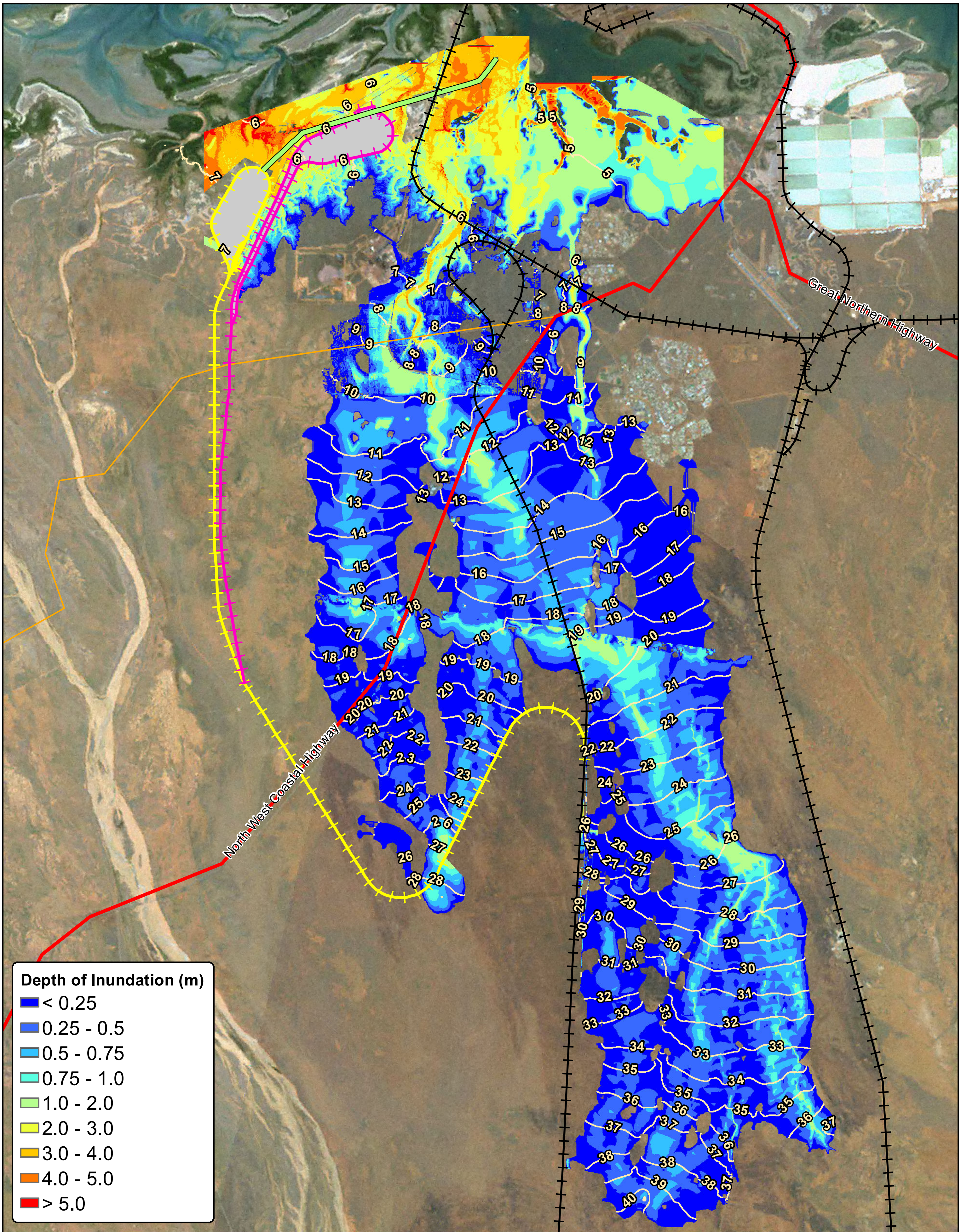
By: S. Bayly

SKM



Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-10 Developed Case 50 year ARI Peak Inundation and Peak Water Level

Job No: QE09828
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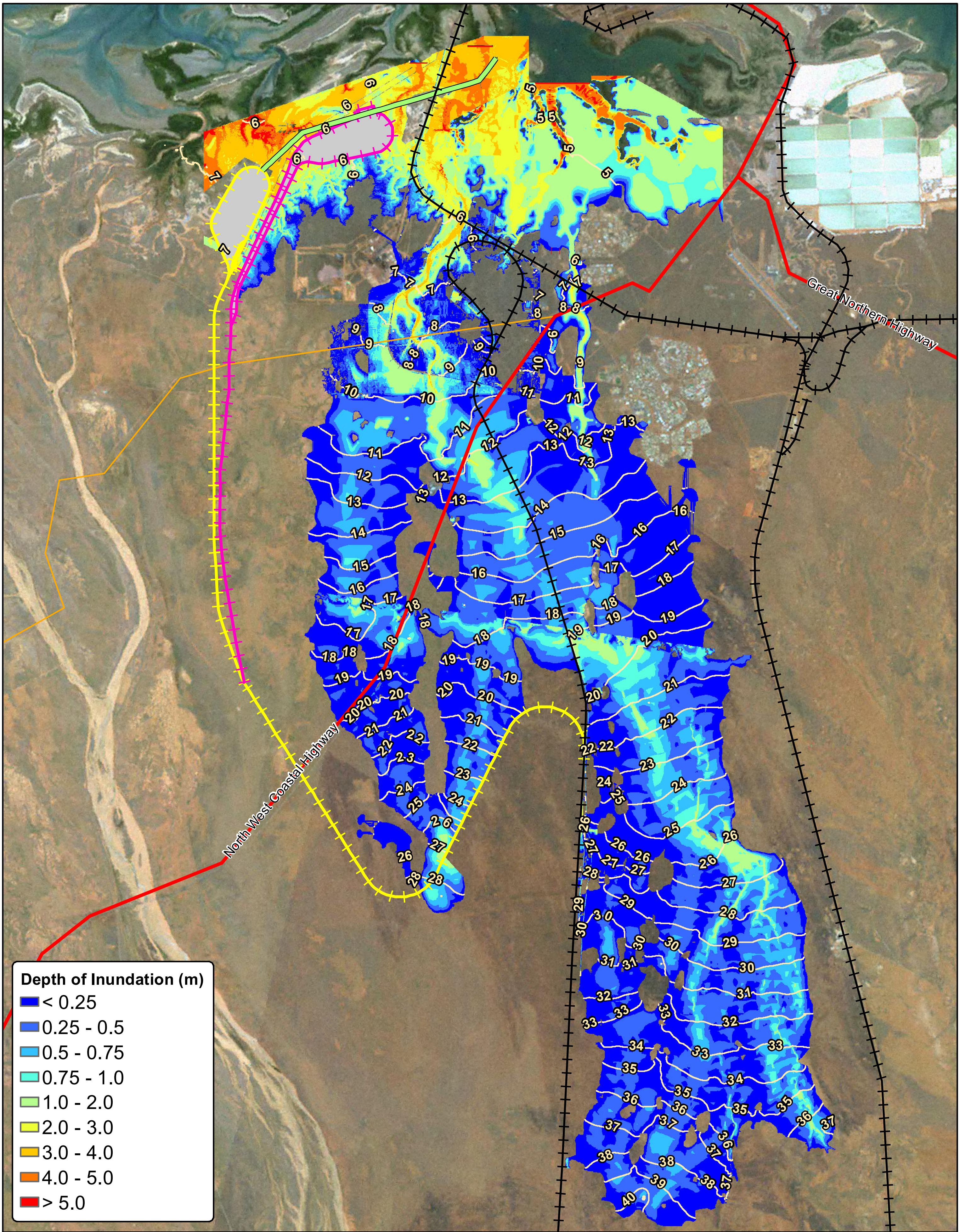
By: S. Bayly

SKM



Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

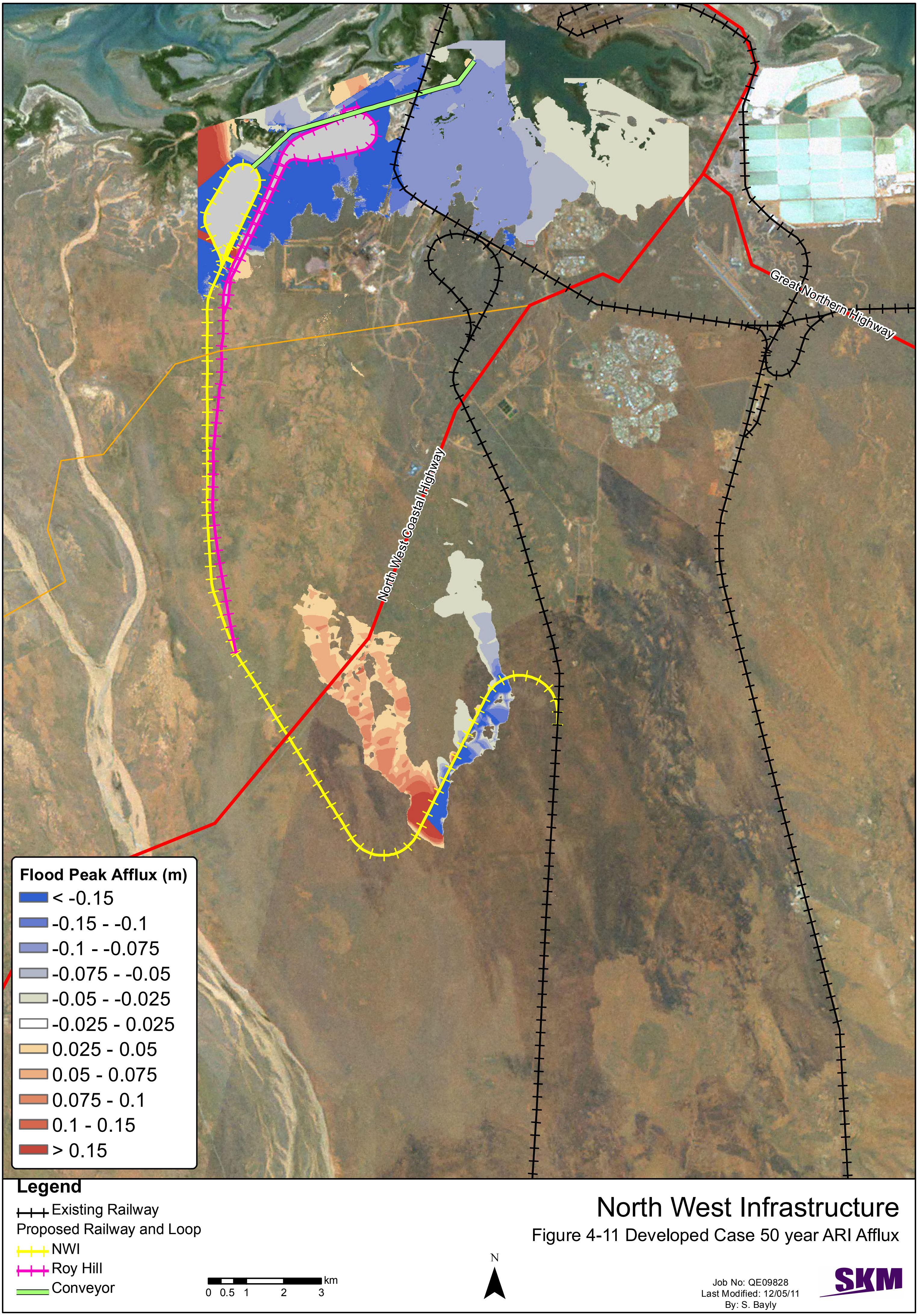
N

North West Infrastructure

Figure 4-10 Developed Case 50 year ARI Peak Inundation and Peak Water Level

Job No: QE09828
Last Modified: 12/05/11
By: S. Bayly

SKM



North West Infrastructure

Figure 4-11 Developed Case 50 year ARI Afflux

Job No: QE09828
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By: S. Bayly

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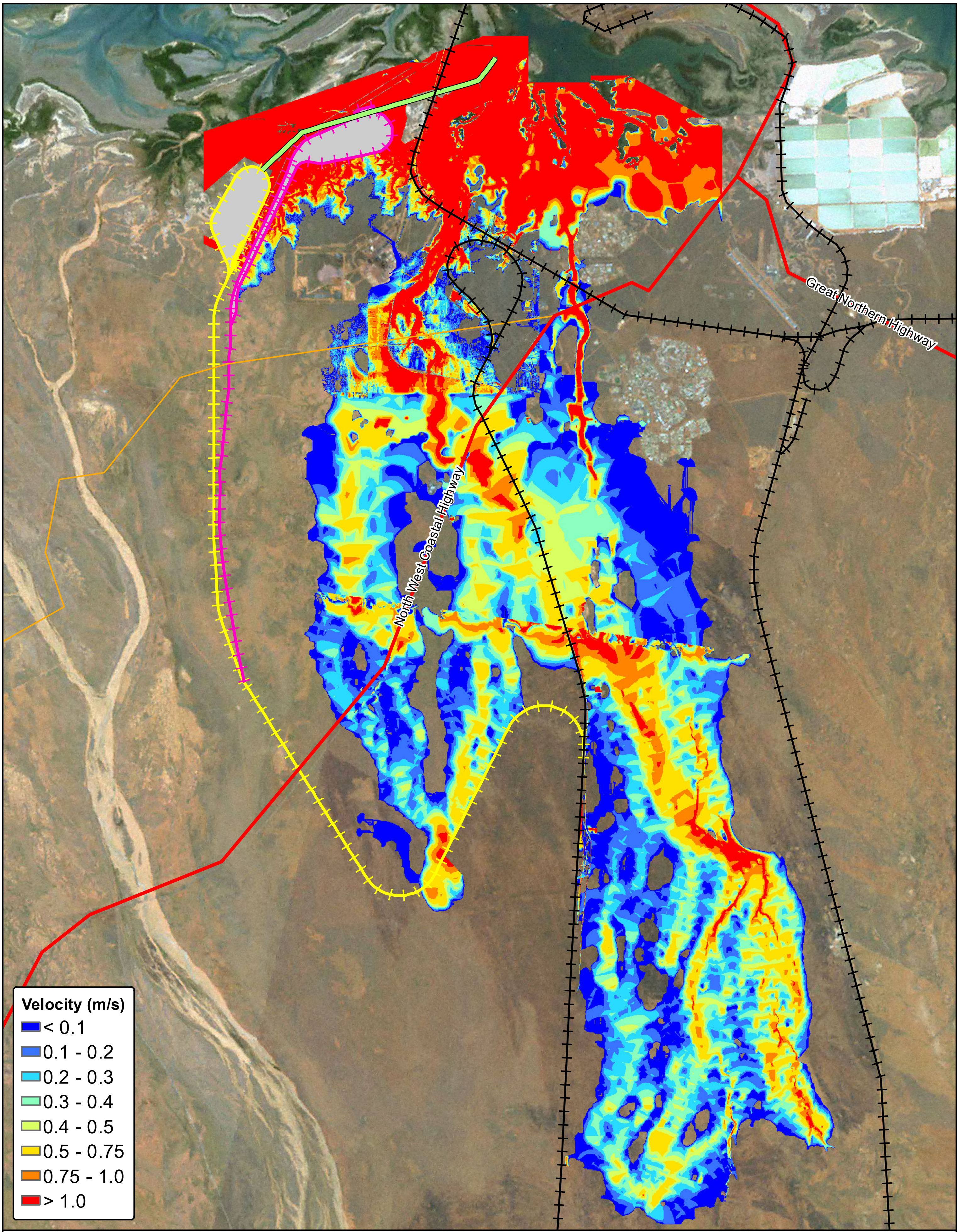


Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

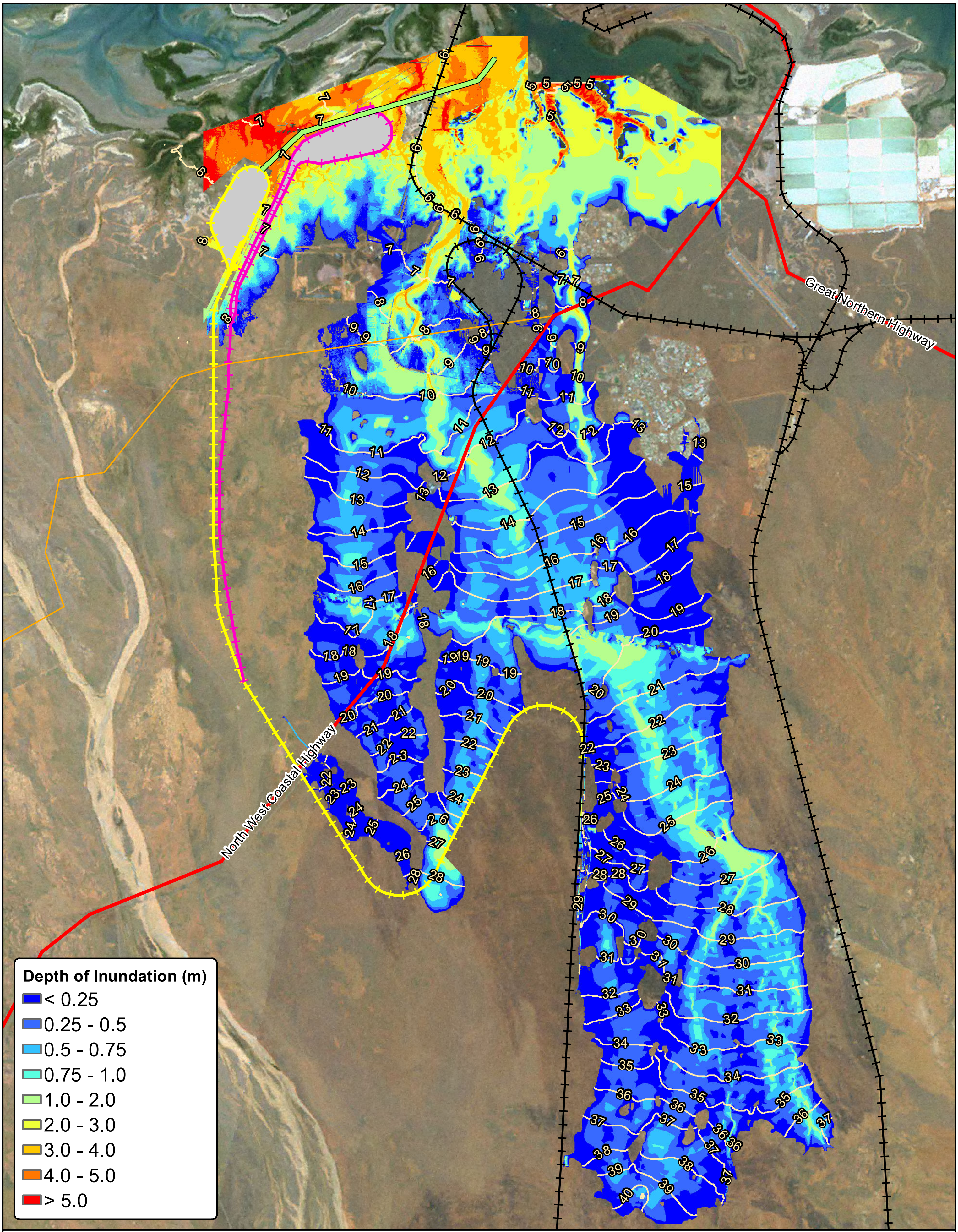
N

North West Infrastructure

Figure 4-12 Developed Case 50 year ARI Peak Velocity

Job No: QE09828
Last Modified: 12/05/11
By: S. Bayly

SKM



Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

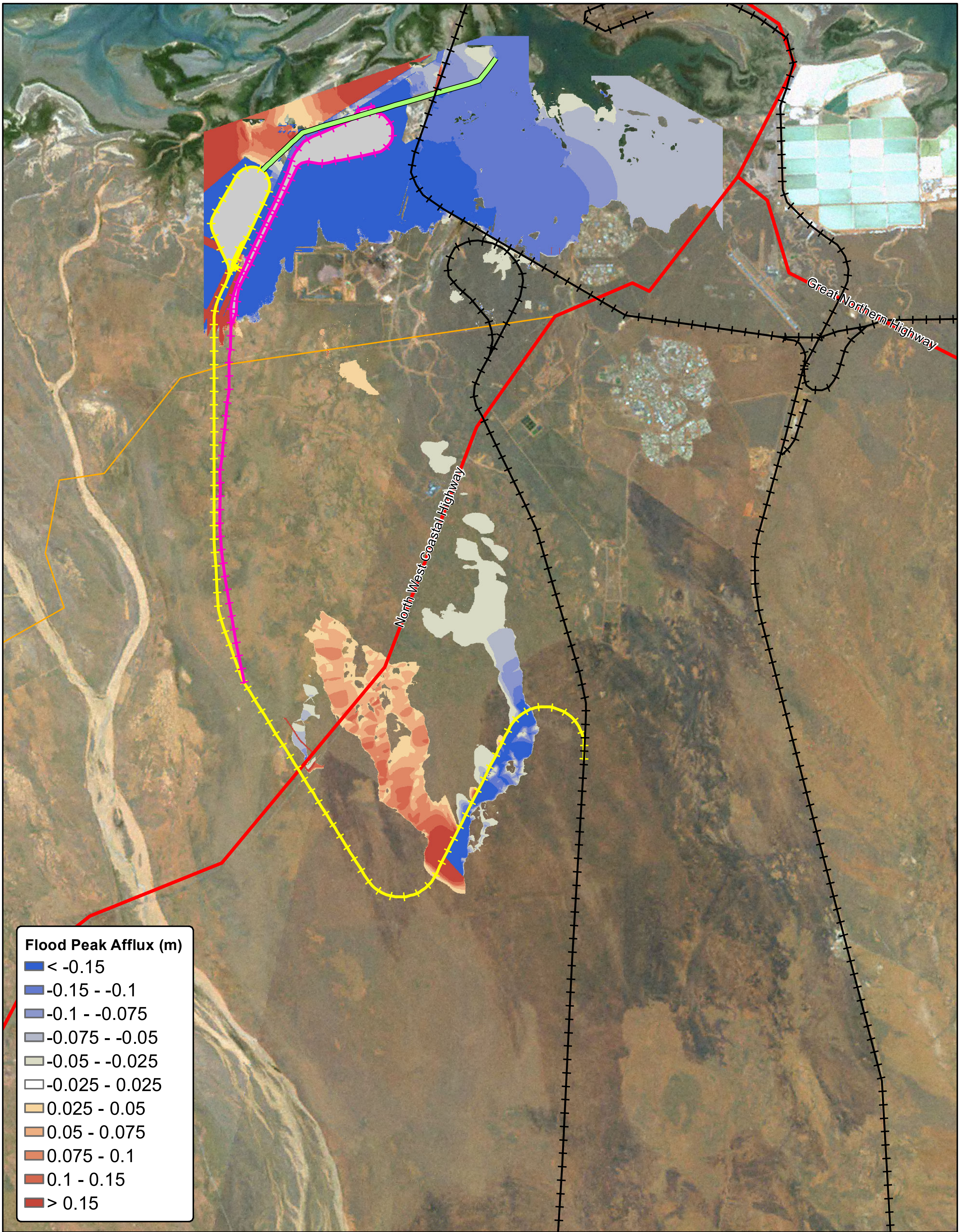
0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-13 Developed Case 100 year ARI Peak Inundation and Peak Water Level

Job No: QE09828
Last Modified: 13/05/11
By: S. Bayly



Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-14 Developed Case 100 year ARI Afflux

Job No: QE09828
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By: S. Bayly

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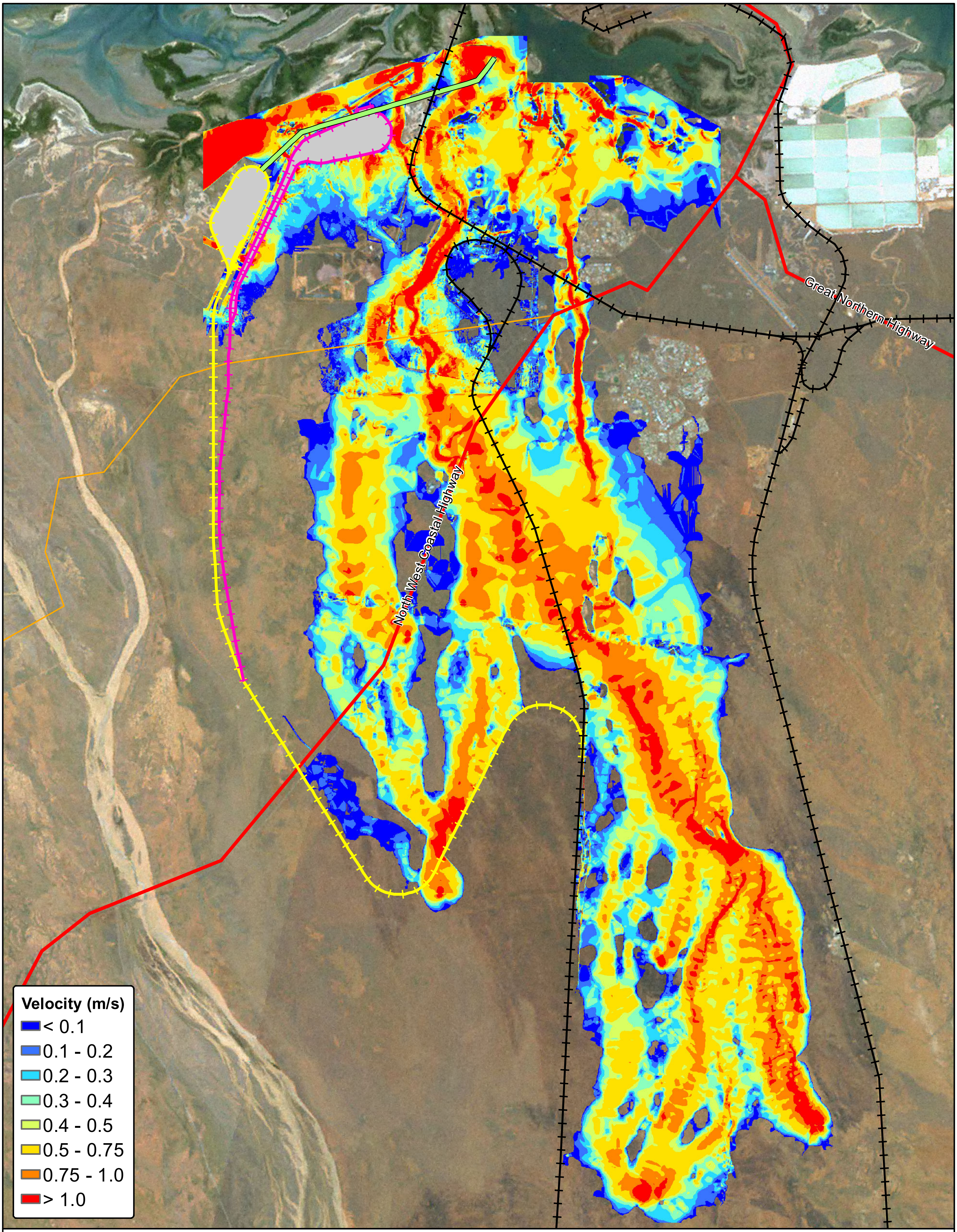

Legend

Existing Railway

Proposed Railway and Loop

NWI

Roy Hill

Conveyor

0 0.5 1 2 3 km

N

North West Infrastructure

Figure 4-15 Developed Case 100 year ARI Peak Velocity

Job No: QE09828
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By: S. Bayly

SKM

5 Assessment of Potential Surface Water Impacts

5.1 Affects on Floodplain Storage

Where the footprint of the railway loop intersects with areas of potential flood inundation under existing conditions, there will be a resulting loss of floodplain storage. The footprint of the NWI railway loop and facilities is 0.15 km², which results in a loss of up to 0.15 km² in floodplain storage. The proposed Roy Hill development would consume a similar area of floodplain. However, most of the proposed railway lines for both developments would be located on more elevated land along the ridge between the South West Creek and Turner River catchments, and therefore much of this floodplain area would only be inundated during a flood event that is more extreme (lower probability) than the 1 in 100 AEP event. The impact of the loss of floodplain storage is considered in conjunction with the effects of impedance to surface flow represented in the hydrological and hydraulic modelling.

5.2 Changes to Rainfall Runoff Rates

Installation of the rail loop, car dumper, access roads, stockyard for iron ore, conveyor, two berth wharf and ancillary infrastructure buildings will cause increases in runoff rates from those that would be observed under the natural catchment conditions, due to increases in the impervious area. This represents 0.03% of the total combined catchment area of South and South West Creeks. This change in the volume of runoff produced is insignificant when compared with the overall total volume of runoff generated from the existing catchments of South and South West Creeks during flood events.

During minor rainfall events there will be additional runoff generated from these impervious areas that would otherwise runoff into areas surrounding the project site. Stormwater management practices, such as detention of runoff from impervious areas to reduce flow velocities and the prospects of scour, may be required to manage these local increases in runoff rates.

5.3 Impedance of Surface Flow

The modelling simulations demonstrated that the largest impact upon runoff is the impedance of surface water flow. If no mitigation measures were put in place, the adopted railway spur line, rail loop and conveyor embankment would constrain the movement of surface water flows.

The spur line crosses South West Creek, which if unmitigated would cause flows from South West Creek to back up and pond along the upstream side of the adopted railway spur line. The mitigation measures to avoid this are to provide adequate culvert capacity through the South West Creek crossing for the adopted railway spur line and to place contour embankments along the upstream side of the adopted railway spur line and to the North of the South West Creek culvert crossing to force South West Creek flows underneath the railway at the crossing location.

If the conveyor were built entirely along an embankment, with no waterway openings, this would impede both flood runoff from South and South West Creeks flowing out to the ocean and storm surge penetrating inland from the ocean. The mitigation measure to maintain impedance of surface water flows at acceptable levels is to build the conveyor structure on pylons to allow for flood and storm surge flows to pass underneath the conveyor.

The proposed development, even with mitigation measures in place, will modify the velocity of flows in the floodplain of South and South West Creeks during flood events, as documented above. Scour protection may be required, particularly in the vicinity of the waterway openings in the conveyor and the culvert through the adopted railway spur line, to control erosion during flood and storm surge events.

5.4 Impacts on Surface Water Quality

Rainfall on iron ore stockpiles and impervious surfaces around the rail loop and conveyor will cause runoff that may contain sediment and low levels of other contaminants. Silt trap and sedimentation basin facilities will be developed to trap sediments washed off the stockpile areas during rainfall events to prevent these materials flowing unmitigated into the receiving environment.

Modelling results indicate the velocities of flows in the floodplain around the site may change as a result of the development, although due to the relatively low level of encroachment of the project site into the floodplain these changes in velocity are expected to be small. Suspended sediment loads of downstream water may be increased if alteration of flow regimes results in scour velocities created downstream of the development. Areas where high flow velocities may be expected should be identified and appropriate measures taken to slow flow and contain sediments on site.

6 Management Measures

During rainfall events, some additional stormwater runoff will be generated from new impervious areas of the of the rail loop, car dumper, access roads, stockyard for iron ore, conveyor, two berth wharf and ancillary infrastructure buildings. On-site stormwater management measures may need to be implemented to detain the runoff produced from these areas and to minimise scour caused by direct runoff from these areas. A review of the DFS indicates a detention basin of 10.3 ha surface area is incorporated for management of runoff within the stockpile area.

The stockpiles, car dumper and conveyor loader will be located above the 1 in 100 AEP flood and storm surge level, to minimise the probability of this infrastructure being inundated. In the event of a large flood or storm surge event that floods the rotary car dumper and the facilities for detaining water released during unloading of the rail cars, some sediment and other contaminants may be released into the environment. During a large flood event (rarer than 1 in 100 AEP), there would be large volumes of water inundating the flood plains of South and South West Creek so that there would be considerable dilution of the potentially contaminated water. Volumes of flood water would be sufficiently large that there would not be expected to be appreciable increases in the concentrations of contaminants.

Culverts will be designed to allow flood flows to pass through the adopted railway spur line at the South West Creek crossing during the 1 in 100 AEP event without causing inundation of the railway line. A regular programme of inspection and maintenance of the culverts would be implemented so that they continue to perform their function in passing flows during flood events. Waterway openings, in the form of bridges or culverts, will be designed in the conveyor embankment to pass the 1 in 100 AEP flood and storm surge event.

Baseline and continuous monitoring of sediment and other pollutants during construction and operation of the facility will be conducted to detect any changes in water quality due to these activities.

7 References

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