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Pike Floodplain Hydraulic Modelling
2012–13

WATER FOR GOOD

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PREPARED: DATE: 12/09/2013

(for acceptance) (Daniel McCullough – Hydrologist)

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(for release) (Brad Hollis – Manager, River Murray Environmental Works)

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Section	Number	Amendment Summary

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1. Introduction and Background

The Pike anabranch complex is located on the eastern bank of the River Murray adjacent to Lock 5. The anabranch supplies water to a local irrigation community and is significantly regulated via man-made embankments and regulating structures. This regulation has resulted in an altered flow path through the anabranch compared to that under natural conditions. Effects of drought, elevated groundwater and altered flow regime have combined to cause a significant decline in the health of the system.

The Pike Floodplain element of the Riverine Recovery Project (RRP) has been instigated to restore the health of the anabranch complex. Planned works under this project include upgrade and/or replacement of existing embankments and infrastructure within the floodplain, which will enable the return of flows through the anabranch towards natural conditions, facilitate fish passage and allow artificial inundation of the floodplain under controlled events while maintaining supply to the Pike irrigation community.

The following report presents the modelling work undertaken under each flow scenario. Each chapter represents a particular scenario, with explanation provided in each case for the purpose of the scenario. Results are presented in both tabular and graphical form where applicable, with analysis of the results and conclusions for each case presented.

2. Model Summary

To facilitate investigations and inform design work related to the planned infrastructure upgrades, hydrodynamic modelling has been undertaken to examine a variety of different flow scenarios within the anabranch complex. Numerical hydrodynamic models originally produced by Water Technology using DHI Software's MIKE FLOOD modelling system were used as the basis for testing these flow scenarios, with modifications to the model implemented where applicable. Model characterisation was reported on internally by Water Technology, with a summary of the report contained in the following sections.

2.1 Numerical Modelling System

The MIKE FLOOD modelling system by DHI Software is a tool that allows coupling of 1 dimensional (1D) with 2 dimensional (2D) hydrodynamic models, providing advantages of each type of model within a single model scheme. For instance, 1D models (developed using the 'MIKE11' module) provide an accurate description of river channels and operating structures, and are less computationally demanding (i.e. shorter run times) than 2D models, while 2D models (developed using the 'MIKE21' module) are useful for providing detailed information of velocity, depths, etc. on floodplains without the need to prescribe flow paths as is the requirement for 1D models. Thus, in-channel is described by the 1D portion of the model while overbank flow is described by the 2D portion. Further information of the MIKE FLOOD modelling system is available on the DHI Software website:

http://www.dhisoftware.com/Products/WaterResources/MIKEFLOOD.aspx

2.2 Model Establishment

A detailed 1D model was developed in MIKE11 describing all the permanent waterways in the Pike anabranch complex and the section of the River Murray between Lock 4 (downstream) and Lock 6 (upstream). The model also includes the Chowilla Creek outlet below Lock 6, allowing model inflow to be specified at both Lock 6 and Chowilla Creek. This model includes all regulating structures, culverts and banks in the anabranch and Lock 5 in the River Murray, with modifiable control schemes defined as applicable. Manning's 'n' hydraulic roughness values of 0.028 were used throughout the waterways and were found to provide good agreement with observed levels.

A 2D model was developed in MIKE21 defining only the Pike anabranch complex area. A grid cell size of 30 m was selected as a trade-off between an adequate description of topographic features and simulation time (N.B. finer grid sizes increase simulation time). Elevation levels greater than 50 m AHD were assigned 'land values', which removed them from computations and decreased simulation time. Manning's 'n' hydraulic roughness values were assigned based on land cover identified in satellite imagery as follows:

- Crops, n = 0.038
- Dense vegetation, n = 0.070
- Medium vegetation, n = 0.050
- Sparse vegetation, n = 0.033

- No vegetation, n = 0.025
- Waterways, n = 0.025

Note that adjustments to the roughness values were made based on the calibration process summarised in Section 2.3.

The models were coupled in MIKE FLOOD to allow overbank flow from the 1D model to transfer to the 2D model and spread across the floodplain, with the following modifications made:

- Sections of the 1D model representing the wider channels in the anabranch were removed (to be represented by the 2D model), including Mundic Lagoon, Tanyaca Creek, and the section of Pike River between the Snake Creek/Pike River confluence and Rumpagunyah Creek.
- Areas in the 2D model defined by the 1D waterways were 'blocked out' by specifying them as 'land values' in order to avoid these areas filling with water and causing double-counting of flow volumes in the model results.
- Linkages between the models were made by defining 44 'lateral' links (i.e. lengthways coupling of 1D waterways to the 2D grid for overbank flow) and 23 'standard' links (i.e. coupling of the start or end of waterways to transfer water from 1D to 2D sections, or vice versa). Links were restricted to primary creeks only to limit computational time of the model (N.B. increasing the number of links results in an increase in model run time).

2.3 Model Calibration

The coupled MIKE FLOOD model was calibrated by measuring model output against actual data, including gauged stream flow data and satellite imagery of inundated areas for a known flow regime. Where differences were encountered, model attributes – in particular Manning's 'n' hydraulic roughness values – were adjusted incrementally in a 'trial and error' solution.

Flow scenarios used for the calibration were developed from actual monitored events, which included:

- A high flow event spanning from September 2000 to January 2001, with Flow to SA (QSA) peaking at approximately 60,400 ML/d; and
- An elevated flow event spanning from September 2005 to January 2006, with QSA peaking at approximately 15,000 ML/d.

Missing level and flow data was encountered at Locks 4 and 5 during the peak of the 2000/01 flow event due to submergence of these locks, requiring the inflow hydrograph to be synthesised from available observed data and the results from an existing Chowilla floodplain model simulating the same event. Satellite imagery of the inundation extent on 13 December 2000 was also available for comparison of results.

The 2005/06 flow event was characterised by flow largely contained within the banks, with the Lock 5 upper weir pool level raised from 16.3 m AHD to approximately 16.8 m AHD. This was reflected in the model by setting Lock 5 as a control structure to dynamically open and close to match the monitored upstream water level throughout the scenario.

2.4 Calibration Results

The calibration results for the 2005/06 event were found to accurately predict water levels at all monitoring sites within approximately 0.1 m, with the exception of sites at Coombs Bridge and Col Col Bank, which did not reflect the levels identified in historical data to the same level of accuracy (i.e. Coombs Bridge level was underestimated by approximately 250 mm while Col Col Bank was overestimated by approximately 100 mm). The differences encountered at these latter sites were suggested to be a result of the uncertainty of the Coombs Bridge geometry, which had changed since 2005, and known issues with the Col Col Bank site including impacts from reeds and heavy siltation.

Calibration of the 2000/01 event focused largely on comparing the modelled results with the inundation extent defined in satellite imagery given the incomplete data available during this event. It was found that the inundation extent in the upper floodplain, comprising the areas upstream of Col Col Bank, corresponded well to the actual inundated area, however the inundation extent in the lower floodplain (below Col Col) was overestimated. This overestimation was attributed to factors such as uncertainty with the River Murray flow, specific operation of Lock 5, and the larger 30 m grid size overestimating flow through some of the smaller creek in the 2D portion of the model. Instability in the 1D-2D linkage between Rumpagunyah and Tanyaca Creek was also encountered at the simulated river flow.

Overall, the model was found to provide accurate representation of flood extents in the Pike system over a range of River Murray flood flows.

3. Scenario 1 – Removal of All Surface Water Management Infrastructure under Flows at Lock 5 of 10,000, 30,000 and 50,000 ML/day

The following section presents the findings of a hydraulic modelling exercise to determine the expected flow distribution in the Pike anabranch complex and associated floodplain under unregulated, “natural” conditions. Natural conditions are defined as conditions with all current infrastructure in and around the floodplain removed, including:

- Locks 4 and 5
- Deep Creek and Margaret Dowling Creek inlet structures
- Coombs Bridge
- Col Col Bank
- Banks B, C, D, E, F, F1, H and G

3.1 Model Simulations

Three flow scenarios were examined to determine the flow distribution and inundation through the floodplain under unregulated, natural conditions, namely at a flow upstream of Lock 5 of (i) 10,000 ML/day, (ii) 30,000 ML/day, and (iii) 50,000 ML/day.

Two separate versions of the model for Pike were used, namely a 1-dimensional (1-D) ‘MIKE 11’ model for in-channel flow, and a ‘MIKE FLOOD’ coupled model including 1-D and 2-D sections for overbank flow. The MIKE FLOOD model was run for each flow scenario to generate inundation maps. Where the flow was shown to be completely in-channel, the MIKE 11 model was subsequently used for data generation in order to simplify data extraction processes. In cases of overbank flow, all data was extracted from the MIKE FLOOD model.

Each model version was set-up identically by removing all weirs, culverts and control structures from the current network, and deleting the channel cross-sections applicable to each feature. Deleted cross-sections were replaced by estimates of cross-sections obtained from sections upstream or downstream of the deleted section. Note that the cross-sections used within the model do not necessarily represent the cross-sections that were present prior to river regulation – in particular at Deep Creek and Margaret Dowling locations – but are assumed to be representative given the lack of readily available bathymetric data collected under natural conditions.

The models require “boundary conditions” to be defined, including upstream flow into the model (i.e. flow upstream of Lock 5) and water level at the outlet of the model (i.e. at Lock 4). As Locks 4 and 5 are removed for the three scenarios tested, an estimate of downstream water level at each flow scenario is required. These estimates were obtained by referring to historical water level data captured at the Lock 4 and 5 sites between 1927 and 1929 immediately preceding lock installation, and relating this data to calculated Flow to South Australia (QSA) using maximum monthly stream discharge. While QSA is not necessarily equivalent to the flow upstream of Lock 5 due to losses, it was considered a sufficient approximation for the purposes of this analysis. Table 1 shows approximate Lock 4 and 5 levels prior to lock construction for each calculated QSA. The historical levels at Lock 5 were used as secondary checks for the model results to ensure the simulation was adequately representing unregulated flow in the River Murray. The long length of river between the area of interest and the model's downstream boundary (26 km) minimises the impact of potential errors in the assumed downstream water level for the specified flows, since the influence of the boundary level decreases with increasing distance from the boundary.

Table 1: Approximate water level at Lock 4 and Lock 5 sites (pre-lock construction) against calculated Flow to South Australia (QSA)

QSA (ML/d)	Lock 4 site Water Level (m AHD)	Lock 5 site Water Level (m AHD)
10,000	10.1	12.5
30,000	11.8	14.4
50,000	12.8	15.6

Outputs of depth, flow, velocity and shear stress under steady state conditions are presented at several defined locations in the floodplain for 10,000, 30,000 and 50,000 ML/d flow scenarios in Tables 2, 3 and 4, respectively, with comparisons made with conditions under current existing conditions (N.B. results presented for existing conditions at 30,000 and 50,000 ML/d are obtained from previous modelling results by Water Technology, which do not include complete results at reporting locations 11 and 17 to 21, and hence only limited results for these locations are included in this report). Velocities within the range of 0.18 to 1.4 m/s are generally considered acceptable for fish habitat (although specific fish species may have narrower acceptable velocity ranges). Velocities within this range are highlighted in each table, and an indication of whether fish habitats under "natural" conditions are "improved" (i.e. existing velocity <0.18 m/s, natural velocity within the 0.18 to 1.4 m/s fish passage range), "declined" (i.e. existing velocity within the fish passage range, natural velocity <0.18 m/s), or "equivalent" (i.e. both existing and natural velocities are within the fish passage range) are designated for each case. Maps of maximum flood inundation extent for each scenario are also produced, which include an indication of flow direction and magnitude where applicable. Numbered reporting locations are shown in Figure 1. The following sections outline each scenario tested.

3.1.1 Scenario 1A – Natural Conditions, Lock 5 Flow = 10,000 ML/d

Inflows to the floodplain are minor to insignificant in the 10,000 ML/d case under natural conditions. All flow is in-channel, and confined to the lower Pike area as shown in Figure 2. The standard inlets to the system at Margaret Dowling and Deep Creek are dry at these flows in the absence of Lock 5. Rumpagunyah Creek is the only significant inlet to the system, however the flows through this creek are only minor (i.e. less than 20 ML/d).

Simulated water level at the Lock 5 site was 12.55 m AHD, corresponding well with the historical level data at 12.5 m AHD (see Table 1).

3.1.2 Scenario 1B – Natural Conditions, Lock 5 Flow = 30,000 ML/d

Most of the permanently inundated waterways (post-lock) are inundated at a Lock 5 flow of 30,000 ML/d, although flows remain in-channel, as shown in Figure 3. Water is present in both Deep Creek and Margaret Dowling by a backwater effect from Mundic Lagoon, however neither creek is flowing. The majority of flow from the River Murray is entering the floodplain through streams downstream of the current Lock 5 site. The highest flow rate is through Rumpagunyah Creek (approximately 1600 ML/d) however flows are also entering the floodplain through current Bank B and C locations (locations 3 and 12), through Swift and Wood Duck Creeks (20 and 21), and through the River Murray inlet immediately upstream of salinity monitoring station A4261023 (19). A combined flow of almost 4800 ML/d is flowing back into the River Murray through the lower Pike River.

The simulated level at Lock 5 was 14.37 mAHD, again corresponding well with historical data (14.4 mAHD; Table 1). Note, this elevation is well below Full Supply Level (FSL) upstream of Lock 5 under existing conditions (16.3 mAHD).

3.1.3 Scenario 1C – Natural Conditions, Lock 5 Flow = 50,000 ML/d

All of the permanently inundated waterways (post-lock) are inundated at 50,000 ML/d, with overbank flow causing inundation of the floodplain, as shown in Figure 4. Flows are entering through Margaret Dowling and Deep Creek at 177 and 272 ML/d, respectively. Note that the water level in Deep Creek is greater than the level in Margaret Dowling (see Table 4) despite the Deep Creek inlet being further downstream on the River Murray; this discrepancy is a result of a steeper gradient along each creek in the absence of regulating structures, while the standard reporting location in Deep Creek is closer to its inlet point than the location in Margaret Dowling.

The highest flows into the floodplain from the main channel are seen at reporting locations 20 (Swift Creek) and 21 (Wood Duck Creek), combining for a flow of approximately 6000 ML/d. The majority of flow into Mundic Lagoon enters via the current Bank C location (location 12), with no flow entering through the current Bank B location (3) due to Margaret Dowling and Deep Creek inflows. Rumpagunyah Creek contributes approximately 1500 ML/d to the floodplain, while flow along Pike River (location 14) is at approximately 1000 ML/d. Total flow exiting the lower Pike is at approximately 10,900 ML/d.

Simulated Lock 5 water level was 15.59 mAHD, aligning almost exactly with historical data (15.6 mAHD; Table 1). Note this elevation is well below FSL upstream of Lock 5 under existing conditions (16.3 mAHD). As an additional check, reported flow over Bank C under “existing” conditions (1,390 ML/d) compares well with recent gauging results conducted at flow conditions of approximately 48,000 ML/d over Lock 5, which yielded a result of 1,374 ML/d.

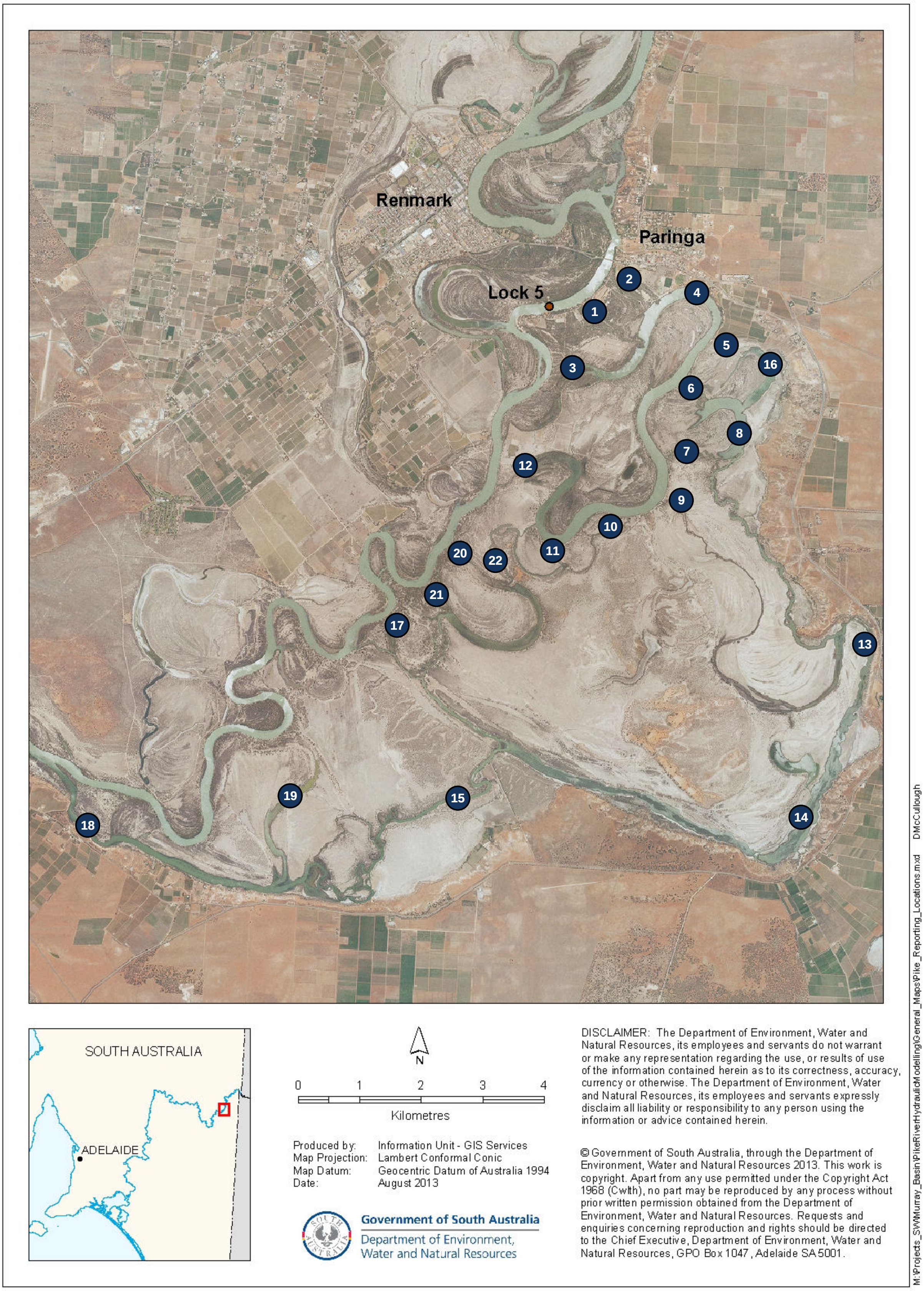

Figure 1: Reporting locations for Pike River hydrodynamic modelling results

Table 2: Water level, discharge, velocity and bed shear stress for existing and natural conditions at Lock 5 flow of 10,000 ML/d (see Figure 1 for reporting locations)

Reporting Location /Stream Name		Existing Conditions – 10,000 ML/d*				Natural Conditions – 10,000 ML/d				Approx change in velocity for fish passage under natural conditions
		h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²	
1	Deep Creek	15.31	182	0.19	0.95	Dry	0	0.00	0.00	Reduction
2	Margaret Dowling	14.82	170	0.16	0.90	Dry	0	0.00	0.00	-
3	Mundic Lagoon - Bank B	14.52	0	0.00	0.00	Dry	0	0.00	0.00	-
4	Mundic Lagoon	14.52	323	0.02	0.00	Dry	0	0.00	0.00	-
5	Mundic Lagoon Outlet 1	14.52	16	0.03	0.01	Dry	0	0.00	0.00	-
6	Mundic Lagoon Outlet 2	14.52	154	0.06	0.02	Dry	0	0.00	0.00	-
7	Mundic Lagoon Outlet 3	14.52	136	0.08	0.05	Dry	0	0.00	0.00	-
8	Upper Pike River	14.52	154	0.01	0.00	Dry	0	0.00	0.00	-
9	Snake Creek - Bank G	14.52	0	0.00	0.00	Dry	0	0.00	0.00	-
10	Tanyaca Creek - Bank F1	14.52	0	0.00	0.00	Dry	0	0.00	0.00	-
11	Bank D	14.52	0	0.00	0.00	Dry	0	0.00	0.00	-
12	Mundic Lagoon - Bank C	14.52	4	0.00	0.00	Dry	0	0.00	0.00	-
13	Pike River	14.40	271	0.04	0.01	Dry	0	0.00	0.00	-
14	Pike River	14.40	254	0.03	0.01	Dry	0	0.00	0.00	-
15	Lower Pike River	13.29	1182	0.09	0.05	11.38	19	0.01	0.00	-
16	Northern Pike Lagoon	14.52	15	0.00	0.00	Dry	0	0.00	0.00	-
17	Rumpagunyah	13.33	674	0.21	0.31	11.78	21	0.06	0.04	Reduction
18	Pike River Outlet	13.26	1385	0.06	0.02	10.87	17	0.002	0.00	-
19	Inlet U/S EC pontoon	13.28	212	0.06	0.05	Dry	0	0.00	0.00	-
20	Swift Creek	13.39	199	0.23	0.48	Dry	0	0.00	0.00	Reduction
21	Wood Duck Creek	13.41	68	0.13	0.19	Dry	0	0.00	0.00	-
22	Tanyaca Ck u/s of Tanyaca Lagoon	13.32	0	0.00	0.00	Dry	0	0.00	0.00	-

*: Values derived from 1-D model, all existing locks and banks in place.

Table 3: Water level, discharge, velocity and bed shear stress for existing and natural conditions at Lock 5 flow of 30,000 ML/d (see Figure 1 for reporting locations)

Reporting Location /Stream Name		Existing Conditions – 30,000 ML/d*				Natural Conditions – 30,000 ML/d				Approx change in velocity for fish passage under natural conditions
		h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²	
1	Deep Creek	15.67	600	0.00	1.02	14.17	0	0.00	0.00	-
2	Margaret Dowling	15.03	400	0.07	2.73	14.17	0	0.00	0.00	-
3	Mundic Lagoon - Bank B	14.91	0.00	0.00	0.00	14.23	802	0.26	0.60	Improvement
4	Mundic Lagoon	14.91	998	0.03	0.03	14.16	798	0.07	0.03	-
5	Mundic Lagoon Outlet 1	0.00	0.00	0.01	0.00	14.16	0	0.00	0.00	-
6	Mundic Lagoon Outlet 2	14.89	429	0.00	0.36	14.16	98	0.05	0.02	-
7	Mundic Lagoon Outlet 3	14.90	513	0.23	0.35	14.16	113	0.11	0.10	Reduction
8	Upper Pike River	14.89	429	0.06	0.00	14.16	103	0.01	0.00	-
9	Snake Creek - Bank G	14.90	0.00	0.00	0.00	14.16	0.17	0.00	0.00	-
10	Tanyaca Creek - Bank F1	14.90	56	0.54	0.00	Dry	0	0.00	0.00	Reduction
11	Bank D	N/A*	N/A*	N/A*	N/A*	14.14	719	0.30	0.68	Unknown
12	Mundic Lagoon - Bank C	14.64	0.00	0.22	0.00	14.18	154	0.13	0.16	Reduction
13	Pike River	14.72	918	0.21	0.29	13.66	211	0.12	0.15	Reduction
14	Pike River	14.70	928	0.24	0.02	13.46	207	0.20	0.63	Equivalent
15	Lower Pike River	14.06	3661	0.00	0.32	13.37	4180	0.31	0.54	Improvement
16	Northern Pike Lagoon	14.89	0.00	0.03	0.00	14.16	0	0.00	0.00	-
17	Rumpagunyah	N/A*	2488*	N/A*	N/A*	13.62	1619	0.41	1.12	Unknown
18	Pike River Outlet	N/A*	N/A*	N/A*	N/A*	12.83	4772	0.22	0.25	Unknown
19	Inlet U/S EC pontoon	N/A*	N/A*	N/A*	N/A*	13.24	600	0.21	0.51	Unknown
20	Swift Creek	N/A*	143*	N/A*	N/A*	14.01	842	0.47	1.64	Unknown
21	Wood Duck Creek	N/A*	0*	N/A*	N/A*	14.04	809	0.48	1.88	Unknown
22	Tanyaca Ck u/s of Tanyaca Lagoon	14.13	116	0.37	0.01	13.78	718	0.35	1.14	Equivalent

*: Values from previous Water Technology modelling results. Limited values reported for locations 11, 17 to 21 as full data for these locations were not included in the original Water Technology reporting.

Table 4: Water level, discharge, velocity and bed shear stress for existing and natural conditions at Lock 5 flow of 50,000 ML/d (see Figure 1 for reporting locations)

Reporting Location /Stream Name		Existing Conditions – 50,000 ML/d*				Natural Conditions – 50,000 ML/d				Approx change in velocity for fish passage under natural conditions
		h m AHD	Q ML/d	v m/s	τ N/m2	h m AHD	Q ML/d	v m/s	τ N/m2	
1	Deep Creek	15.71	600	0.00	0.95	15.33	272	0.28	0.64	Improvement
2	Margaret Dowling	15.28	400	0.07	1.31	15.08	177	0.13	0.12	-
3	Mundic Lagoon - Bank B	15.23	1	0.00	0.00	15.06	0	0.00	0.00	-
4	Mundic Lagoon	15.23	993	0.03	0.03	15.06	372	0.03	0.01	-
5	Mundic Lagoon Outlet 1	0.00	0.00	0.03	0.00	15.06	0	0.00	0.00	-
6	Mundic Lagoon Outlet 2	15.22	582	0.04	0.38	15.06	350	0.08	0.05	-
7	Mundic Lagoon Outlet 3	15.24	493	0.25	0.17	15.06	630	0.22	0.37	Equivalent
8	Upper Pike River	15.21	581	0.05	0.00	15.05	442	0.02	0.00	-
9	Snake Creek - Bank G	15.23	0.00	0.01	0.00	15.06	66	0.05	0.06	-
10	Tanyaca Creek - Bank F1	15.21	489	0.39	0.01	15.05	105	0.02	0.01	Reduction
11	Bank D	N/A*	N/A*	N/A*	N/A*	15.05	467	0.10	0.06	Unknown
12	Mundic Lagoon - Bank C	15.28	1390	0.23	0.01	15.22	1264	0.33	0.76	Equivalent
13	Pike River	15.06	1174	0.15	0.31	14.92	1099	0.20	0.19	Improvement
14	Pike River	15.04	853	0.49	0.01	14.90	1091	0.05	0.01	Reduction
15	Lower Pike River	14.93	9905	0.00	1.25	14.80	9298	0.43	0.93	Improvement
16	Northern Pike Lagoon	15.21	0.00	0.14	0.00	15.05	0	0.00	0.00	-
17	Rumpagunyah	N/A*	1028*	N/A*	N/A*	14.96	1538	0.20	0.21	Unknown
18	Pike River Outlet	N/A*	N/A*	N/A*	N/A*	14.36	10881	0.64	1.42	Unknown
19	Inlet U/S EC pontoon	N/A*	N/A*	N/A*	N/A*	14.56	1443	0.08	0.05	Unknown
20	Swift Creek	N/A*	4052*	N/A*	N/A*	15.13	3794	0.58	1.07	Unknown
21	Wood Duck Creek	N/A*	2436*	N/A*	N/A*	15.25	2259	0.50	1.56	Unknown
22	Tanyaca Ck u/s of Tanyaca Lagoon	15.16	1230	0.36	0.14	15.03	664	0.08	0.05	Reduction

*: Values from previous Water Technology modelling results. Limited values reported for locations 11, 17 to 21 as full data for these locations were not included in the original Water Technology reporting.

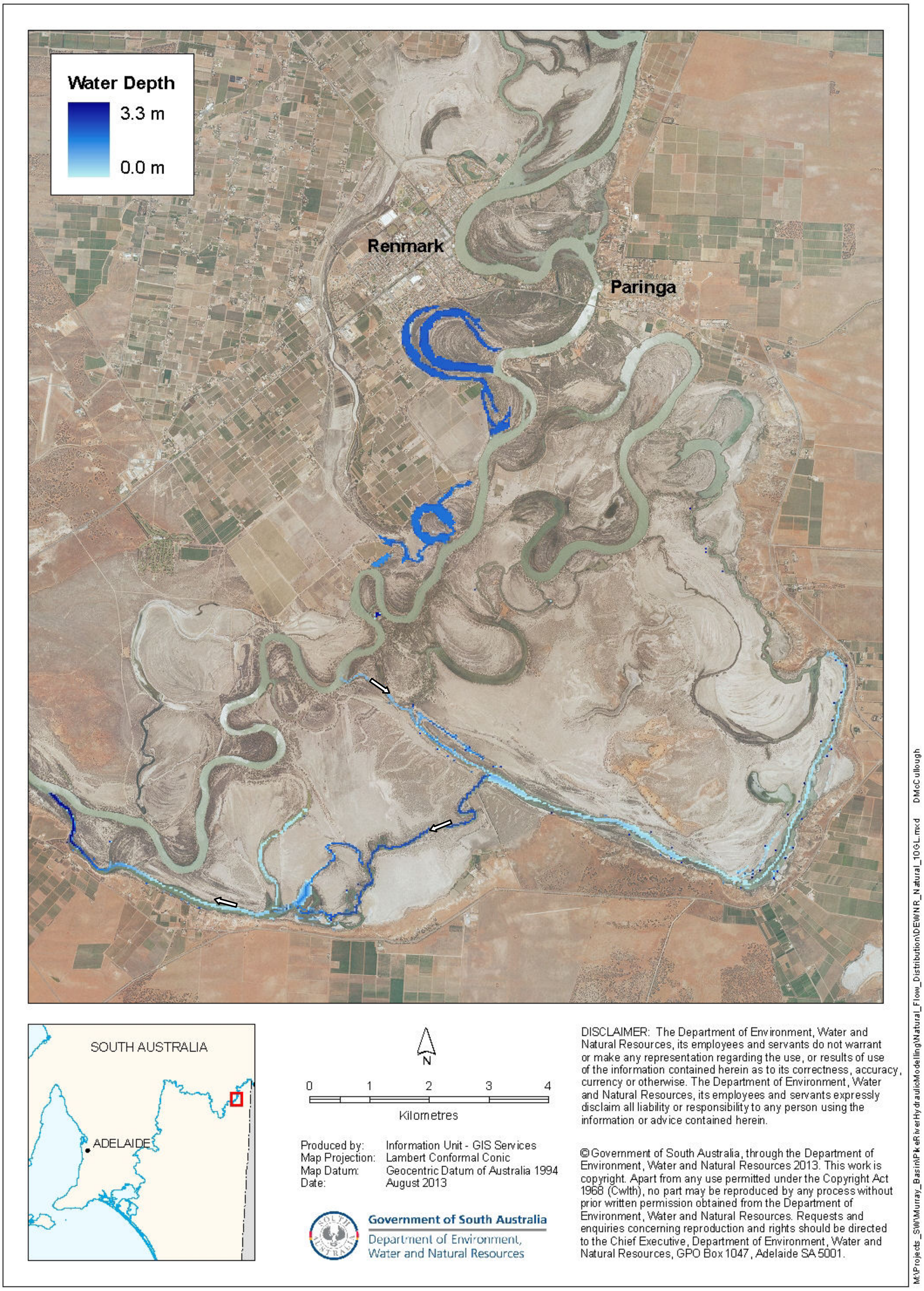

Figure 2: Inundation extent for 10,000 ML/d in Renmark-Paringa reach, unregulated flow (N.B. Lock 5 removed for scenario). Flow direction shown by arrows.

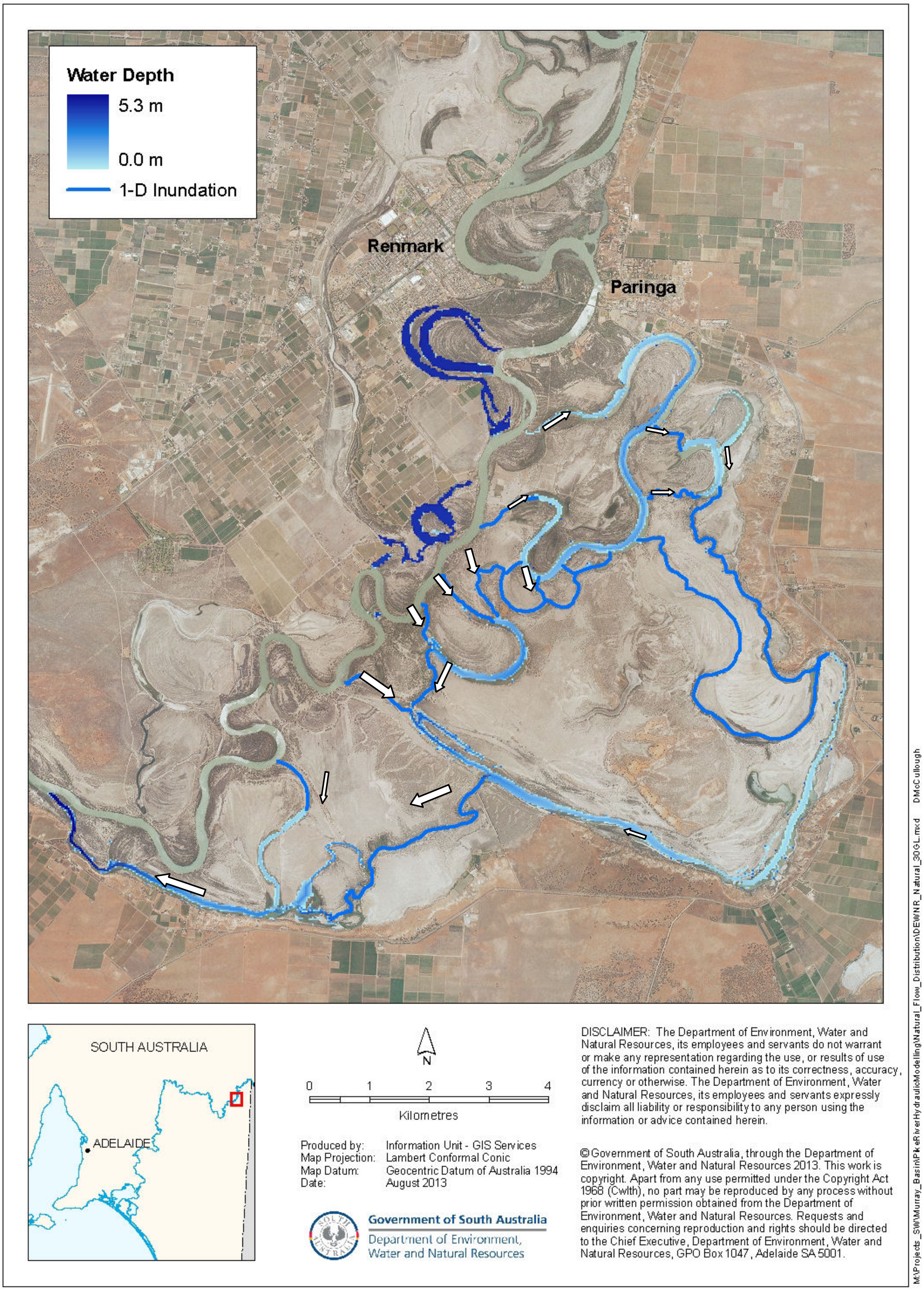

Figure 3: Inundation extent for 30,000 ML/d in Renmark-Paringa reach, unregulated flow (N.B. Lock 5 removed for scenario). Flow direction shown by arrows, with approximate flow magnitude indicated by arrow size.

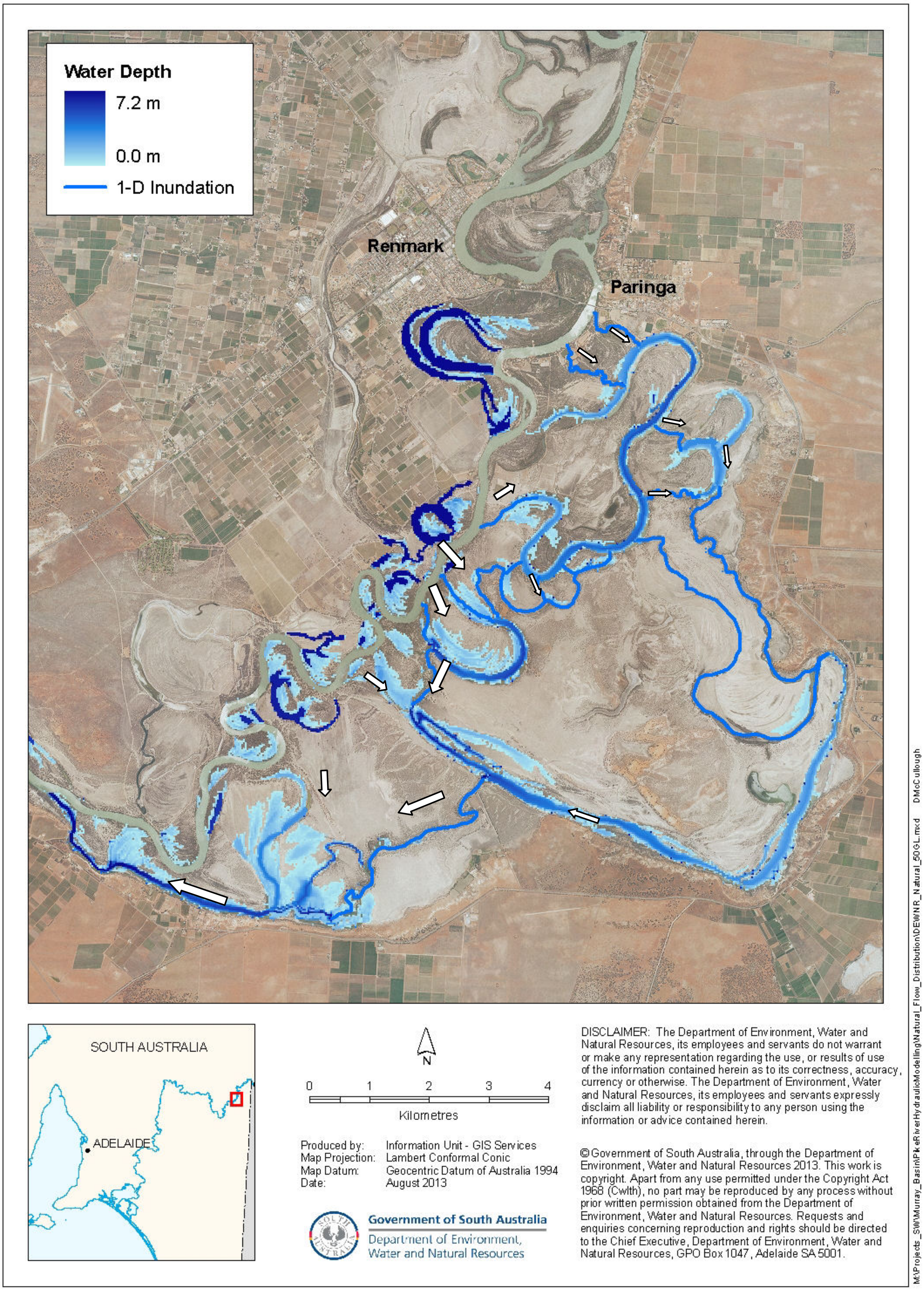

Figure 4: Inundation extent for 50,000 ML/d in Renmark-Paringa reach, unregulated flow (N.B. Lock 5 removed for scenario). Flow direction shown by arrows, with approximate flow magnitude indicated by arrow size.

3.2 Summary

The removal of all regulating structures from the River Murray alters the hydraulic gradient in the main channel, which in turn impacts on the locations and rates of inflow into the Pike anabranch complex and associated floodplain that exist under current, regulated conditions. Flow through the "usual" inlets upstream of Lock 5 at Deep Creek and Margaret Dowling was not encountered at either 10,000 or 30,000 ML/d Lock 5 flow, with the majority of water entering through inlets below the Lock 5 site.

Rumpagunyah is the only inlet into the system at Lock 5 flow of 10,000 ML/d, with much of the floodplain dry or not flowing. Throughflow through the Pike floodplain (indicated by the flow exiting the lower Pike River) is at a minor rate of less than 20 ML/d.

Under "natural" conditions, at 30,000 ML/d at Lock 5, flow is entering the Pike anabranch complex and associated floodplain through all of the major inlets other than Deep Creek and Margaret Dowling. Compared to "existing" conditions, flow is higher through:

- Bank B Creek (800 ML/d "natural" compared to 0 ML/d "existing"),
- Tanyaca Creek U/S of Tanyaca Lagoon (718 ML/d compared to 116 ML/d),
- Swift Creek (842 ML/d compared to 143 ML/d), and
- Wood Duck Creek (809 ML/d compared to 0 ML/d),

while flow is reduced through:

- 'Mundic Lagoon Outlet 2' (98 ML/d compared to 429 ML/d),
- 'Mundic Lagoon Outlet 3' (113 ML/d compared to 513 ML/d),
- Upper Pike River (103 ML/d compared to 429 ML/d), and
- Pike River U/S of the current Col Col site (207 ML/d compared to 928 ML/d).

This flow redistribution is consistent with the intended purpose of the "crude" embankments, specifically to divert water from its "natural" flow path through Tanyaca Creek (via Bank D Creek) to instead augment flow into the Upper Pike River from 'Mundic Lagoon Outlet 2' to support the irrigation interests on the Pike highland upstream of Col Col.

Overbank flow with inundation of lower elevation areas only occurs at 50,000 ML/d, with flow entering the anabranch complex and floodplain through all the major inlets including (to a minor extent) Deep Creek and Margaret Dowling. Much of the inundated area is located in the Lower Pike River. Overall differences in flow distribution between "natural" and "existing" conditions are not as significant as for the 30,000 ML/d flow at Lock 5 scenario, however on a stream by stream basis the main differences include higher flows through:

- Rumpagunyah (1538 ML/d "natural" compared to 1028 ML/d "existing"), and
- 'Mundic Lagoon Outlet 3' (630 ML/d compared to 493 ML/d),

and reduced flows within:

- Mundic Lagoon (372 ML/d compared to 993 ML/d),
- 'Mundic Lagoon Outlet 2' (350 ML/d compared to 582 ML/d),

- 'Tanyaca Creek U/S of Tanyaca Lagoon' (664 ML/d compared to 1230 ML/d), and
- through the lower Pike River (9298 ML/d compared to 9905 ML/d).

The results suggest that the effects of the artificial embankments on flow redistribution between "existing" and "natural" conditions are most apparent at 30,000 ML/d at Lock 5, in which Margaret Dowling and Deep Creek are not yet flowing, while less of a flow distribution difference exists at 50,000 ML/d at Lock 5.

4. Scenario 2 – Salinity Minimisation Scenario: Alternate Operation of Draining Phase

The following section presents the findings of a modelling exercise to examine the impacts on the Pike floodplain of the operation of proposed environmental regulator structures, with upgraded regulating structures in place, over a period of approximately 120 days at a flow at Lock 5 of 10,000 ML/d. The regulators are operated to minimise salinity impacts on irrigators throughout the Pike River (upstream and downstream of Col Col), using Col Col as the predominant outflow location (the remainder flowing out through Tanyaca) during the draining phase of the operational regime.

4.1 Model Simulation

The operational scenario tested includes the following operating regime:

- Lock 5 flow of 10,000 ML/d with Lock 5 at an upstream water level of 16.8 mAHD (0.5 m above normal operating level) for the entire operational period.
- A total of 1,000 ML/d flowing into the Pike system at Deep Creek (600 ML/d) and Margaret Dowling (400 ML/d) during operation.
- The Bank B complex and Bank C remain closed during operation to negate the requirement for fishway structures at these locations during the draining phase of the operational regime.
- **Filling phase:** Flows over the environmental regulating structures are held at 250 ML/d over Col Col and 50 ML/d over Tanyaca Creek structures. This restricts the rate of water level rise immediately upstream of the regulators to less than 10 cm/d, allowing water levels upstream of Col Col to be raised to 16.4 mAHD over a period of approximately 4 weeks.
- **Holding phase:** Impounded water levels are held for approximately 40 days. During this phase, normal variations in flood levels are replicated by varying flows over each environmental regulator to a total flow of 1,000 ML/d (e.g. 600 ML/d over Col Col and 400 ML/d over Tanyaca Creek regulators, and vice versa).
- **Draining phase:** Water level is lowered at a rate of less than 10 cm/d by allowing 50 ML/d over Tanyaca and 1450 ML/d over Col Col regulators, equivalent to 150% of the total inflow through Deep Creek and Margaret Dowling (note that maximum concept design flows through Tanyaca and Col Col regulators are 1,300 ML/d and 3,800 ML/d, respectively). Approximately 5 to 6 weeks are required to draw down the level immediately upstream of Col Col to the ‘normal’ operating level of 14.33 mAHD (i.e. a 2 m reduction in head from full inundation extent).

Outputs generated from the scenario include:

- Maps of flow distribution at 10 day intervals for the duration of the operational management scenario, including a map of the maximum inundation extent.
- Water balance for the scenario.
- Tabulated data of depth, velocity and shear stress at the various reporting locations shown in Figure 1.

- Velocity distribution maps at selected times during regulator operation.
- The outputs are compared with previous modelling results by Water Technology under similar conditions. Note that the previous results are provided as indicative only, as these relate to a Lock 5 flow of 30,000 ML/d; while flow conditions upstream of the blocking bank should be equivalent, some change in flow distribution downstream of the blocking bank is expected.

The main differences in the previously modelled scenarios compared to the current scenario relate mainly to the draining phase of operation, whereby Banks B and C were previously used to assist in draining the floodplain (N.B. these are not opened in the current scenario), and relative flows through Col Col and Tanyaca regulators were more evenly spread (i.e. the majority of flow passes through Col Col regulator in the current scenario to maximise salinity dilution for irrigators upstream and downstream of Col Col on the Pike River. The period of operation also differs due to the draining phase methodology, draining completely after approximately 85 days of operation (from commencement of the filling phase) in the previous scenarios compared with approximately 110 days in the current scenario.

To ensure consistency of the results with previous analyses, the analysis methodology used in the previous scenarios is followed where possible in the current scenario, which includes:

- The water balance over the operational management scheme is considered only up until the system is suitably drained, and does not account for water passing through the system beyond this point. The water balance also only considers water upstream of the blocking alignment. The differences between the operational management periods used are accounted for by calculating outflow and loss volumes as a percentage of inflow volumes.
- Water losses during operation are characterised by mean daily evaporation data (Class A evaporation pan) from the nearest weather station at the Loxton Research Centre (Bureau of Meteorology, 2012). Evaporation rates are applied during post-processing of data to enable reporting of mass lost to evaporation (N.B. system mass loss is not calculated by the model when included in the model scheme).

Some minor differences are possible in post-processing techniques used between the current and previous scenarios (e.g. due to improved model software functionality in recent versions, etc.), which may result in minor differences in results. Minor differences in results may also be incurred from minor changes that were implemented to the model setup in an attempt to improve model stability and reduce potential mass errors from the previous model version, although these changes are not expected to significantly alter results.

4.2 Results

Discharges and water levels during the operational management scenario are illustrated in Figure 5. Note that the Tanyaca regulator remains closed for a period of 1 day at the commencement of the scenario to improve initial flow stability, however in practice this delayed opening is not necessarily required. Flow through Col Col regulator during the draining phase is maintained at 1,450 ML/d until approximately Day 108 of operation, after which the flow is reduced to maintain a ‘normal’ operational level of approximately 14.33 m AHD upstream of the regulator. Flow through Tanyaca environmental regulator is maintained at 50 ML/d for the entire draining phase and post-operation.

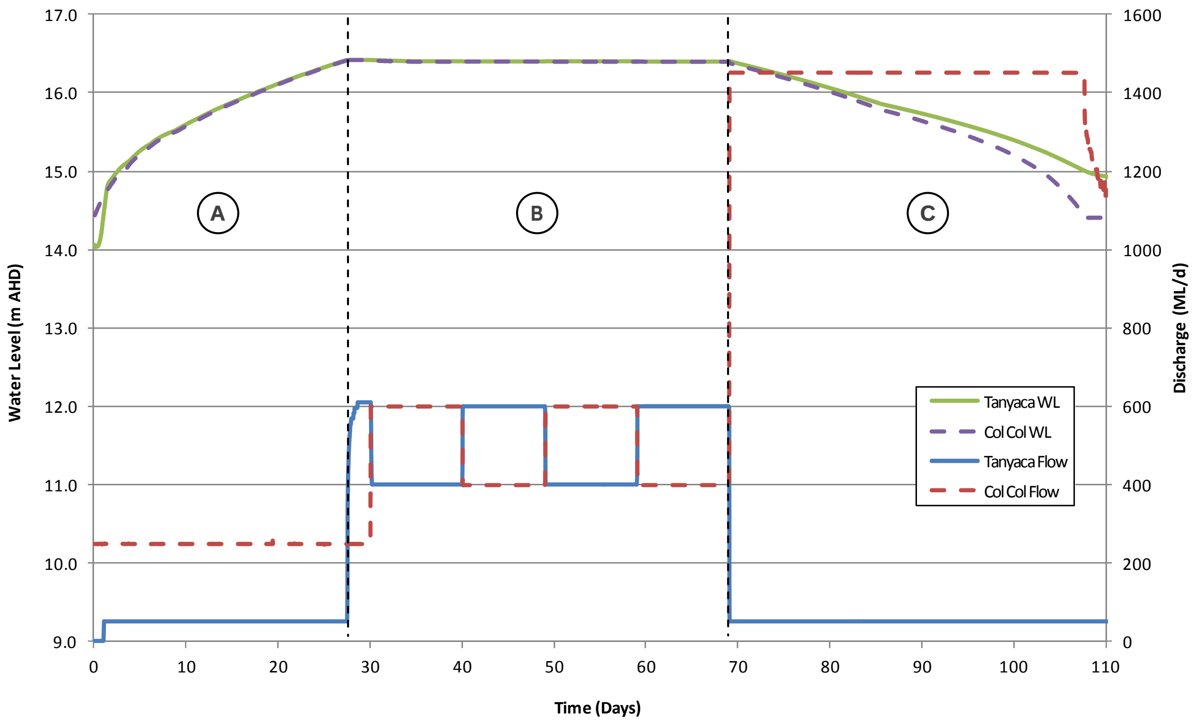

Figure 5: Water level and discharge at Tanyaca and Col Col regulators during operation. A – Filling Phase, B – Holding Phase, C – Draining Phase.

The progress of inundation upstream of the blocking bank is demonstrated in Figure 6, with trends of inundated volume, area, and volume of loss shown. The plot shows a maximum inundated area of approximately 1,700 ha with a corresponding volume of approximately 22,400 ML. Losses are relatively minor, with a maximum daily loss of under 130 ML. Figure 7 shows a similar plot of results from previous Water Technology modelling. Note the volume and area during the draining phase reduces at a more rapid rate under the previous scenario, which can be partly attributed to the additional draining capacity provided by Banks B and C.

A comparison of Figure 6 and Figure 7 indicates a difference of approximately 3% in the maximum inundation extent exists between current and previous scenarios. Given the filling and holding phases between current and previous scenarios are largely identical, and inflows upstream of the blocking bank are the same regardless of flow at Lock 5, this difference appears to be a result of minor differences in the method of data extraction and analysis between the scenarios rather than any significant difference in operation.

Total water balance over the managed inundation scenario is presented in Table 5, with comparison to existing (pre-upgrade) conditions and previous scenario results calculated by Water Technology. Accounting for the differences in period of operation, similarities exist between outflow (88.7% of total inflow compared with 89.4% in the previous scenario) and losses (8.8% current to 8.7% previous scenarios). Storage change (i.e. inundated volume post-operation minus initial volume) is higher in the current scenario (2,700 ML) compared with the previous scenario (1,600 ML), which may be a result Bank B and C remaining closed during the drainage phase, thereby preventing additional drainage that was achievable in the previous scenario. In both scenarios, losses are significantly higher (i.e. 30-40% higher) than those resulting from existing conditions as a result of the significantly larger area of inundation under the managed operational scenarios.

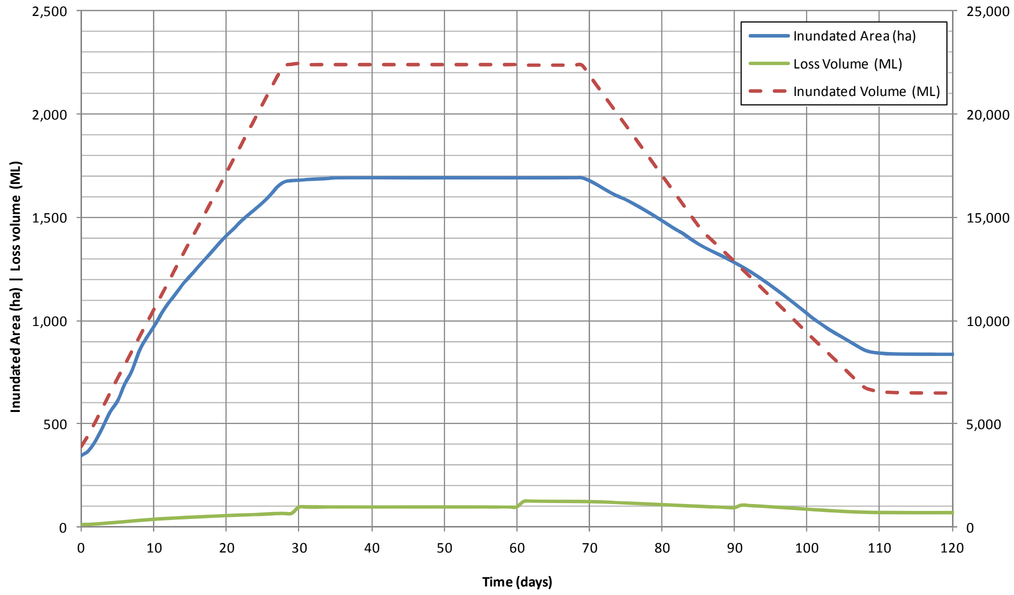

Figure 6: Area and volume inundated upstream of blocking bank for management scenarios, including volume of losses.

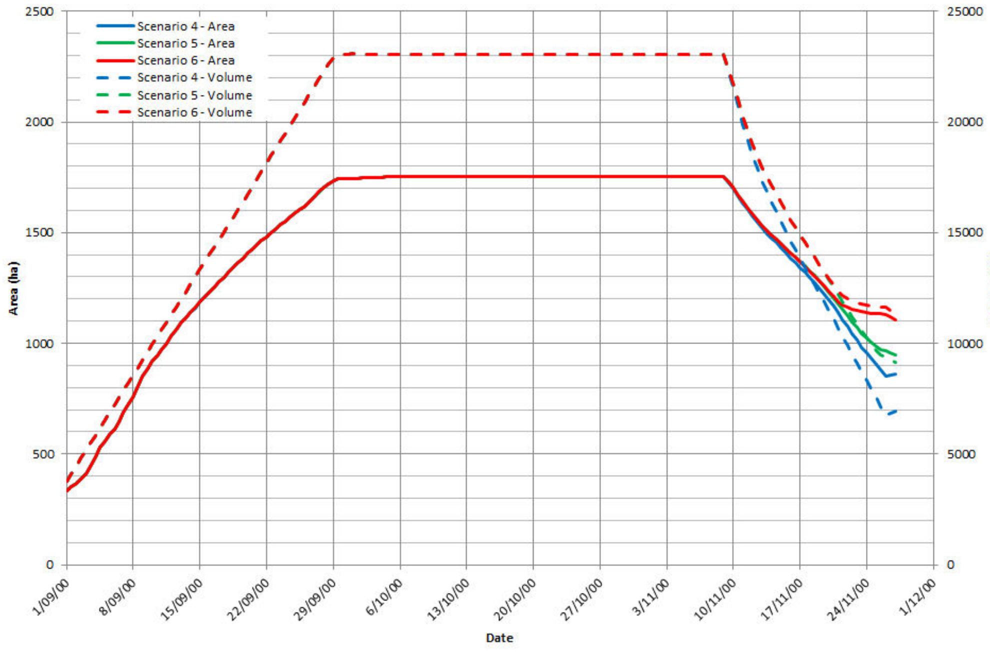

Figure 7: Results from previous Water Technology scenarios for volume and area inundated upstream of the blocking bank for Scenarios 4 (30,000 ML/d), 5 (40,000 ML/d) and 6 (50,000 ML/d).

Table 5: Comparison of total water balances upstream of blocking alignment from start of filling phase to end of draining phase. Bracketed values indicate percentage of total inflow volume.

Water Balance Component (from start of filling phase to end of draining phase)	Total Volume – Existing Conditions[1] ML (% of inflow)	Total Volume – Banks B and C for drainage[1] ML (% of inflow)	Total Volume – Banks B and C closed ML (% of inflow)
Inflow	85,000	85,000	110,000
Outflow	78,900 (92.8%)	76,000 (89.4%)	97,600 (88.7%)
Losses	6,100 (7.2%)	7,400 (8.7%)	9,700 (8.8%)
Storage Change (final – initial volume)	0	1,600	2,700
Total Loss	6,100 (7.2%)	9,000 (10.6%)	12,400 (11.3%)

[1] Values obtained in previous Water Technology modelling – results are identical regardless of flow at Lock 5 assigned.

Figure 8 through to Figure 20 present the dynamic progress of inundation throughout the current scenario (in 10 day increments), from initial conditions to 120 days of operation. Maximum inundation extent is represented by maps of Day 40 (Figure 12) to Day 60 (Figure 14).

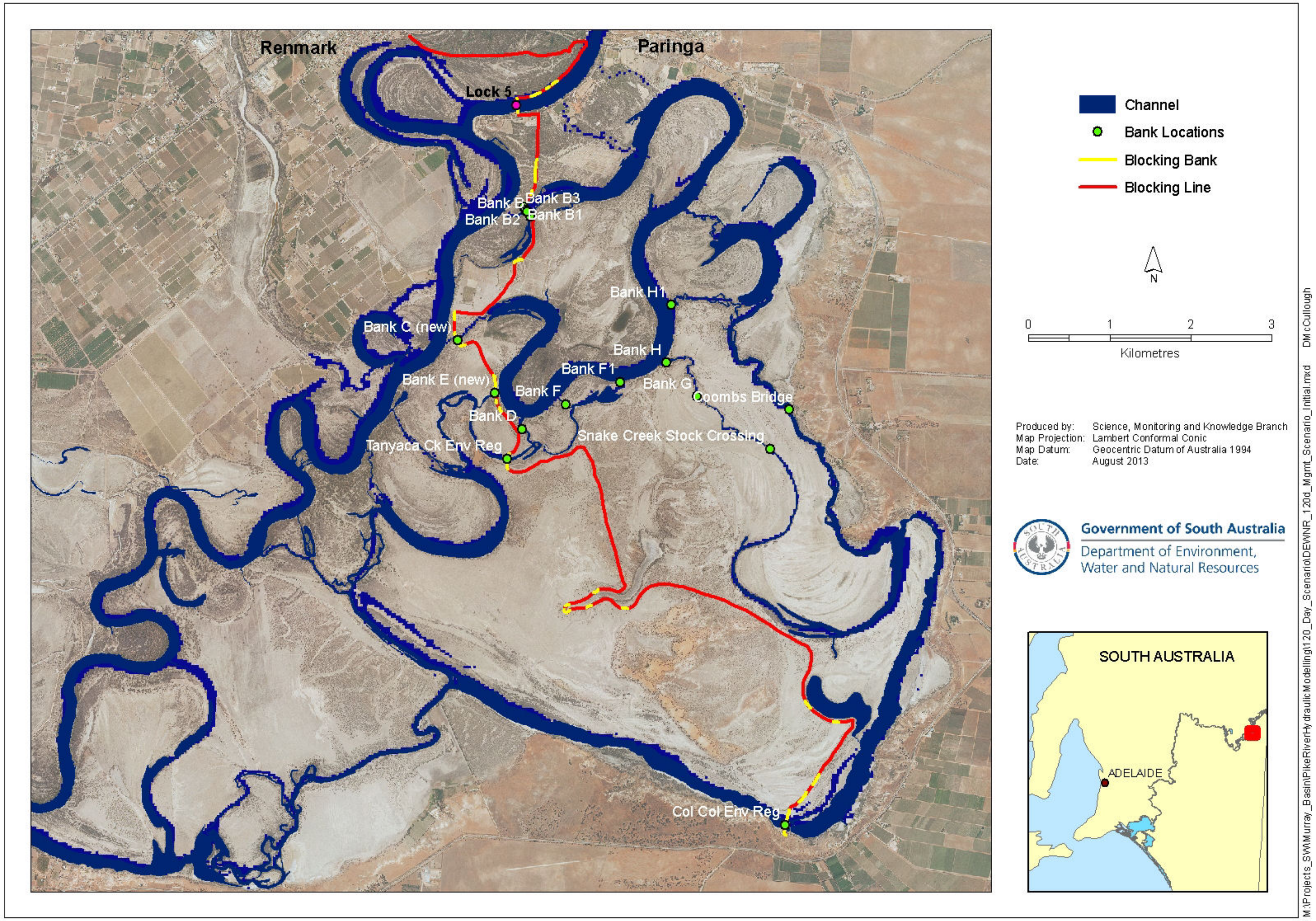

Figure 8: Inundation extent during Pike environmental watering, initial conditions.

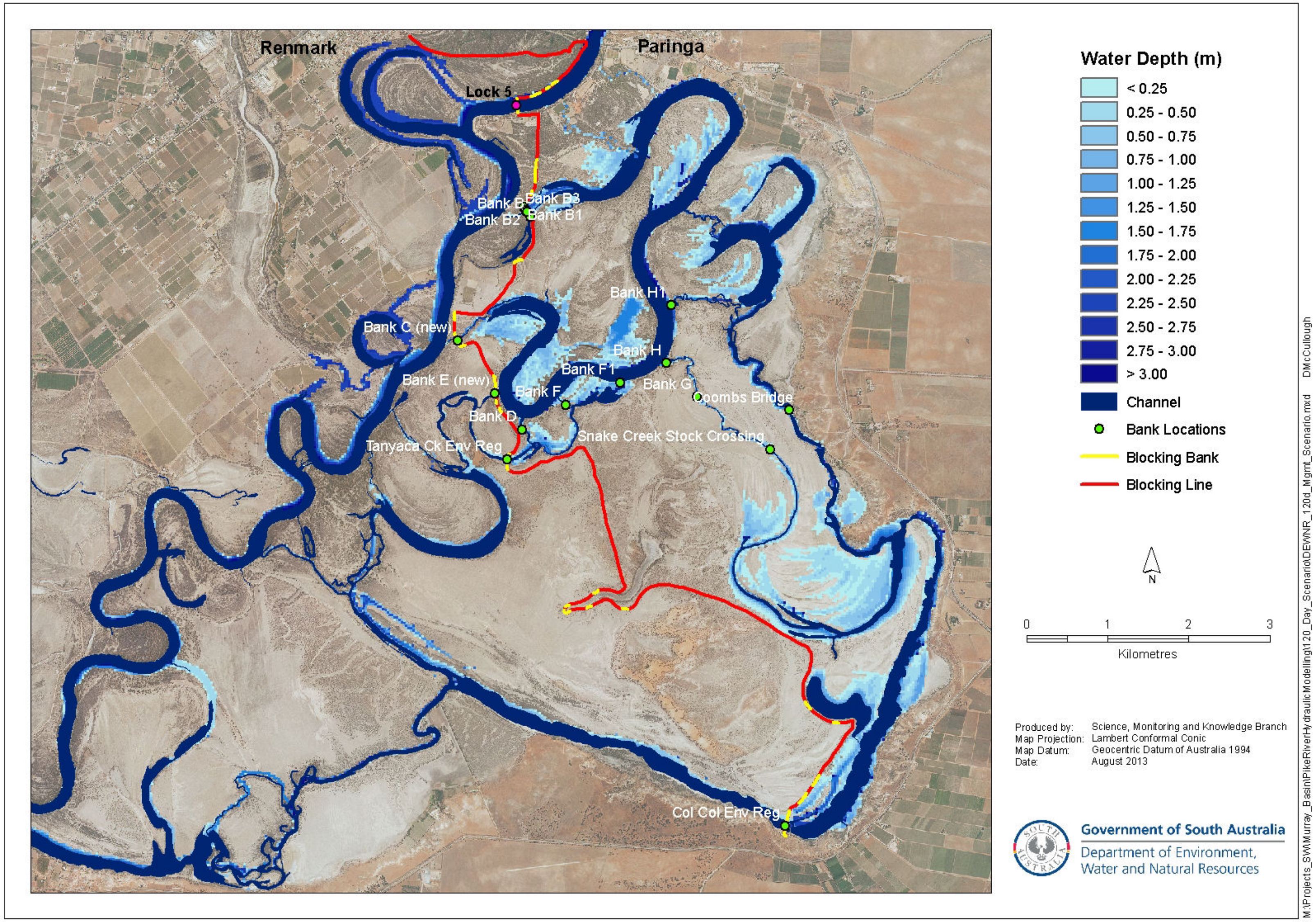

Figure 9: Inundation extent during Pike environmental watering, Day 10 of operation.

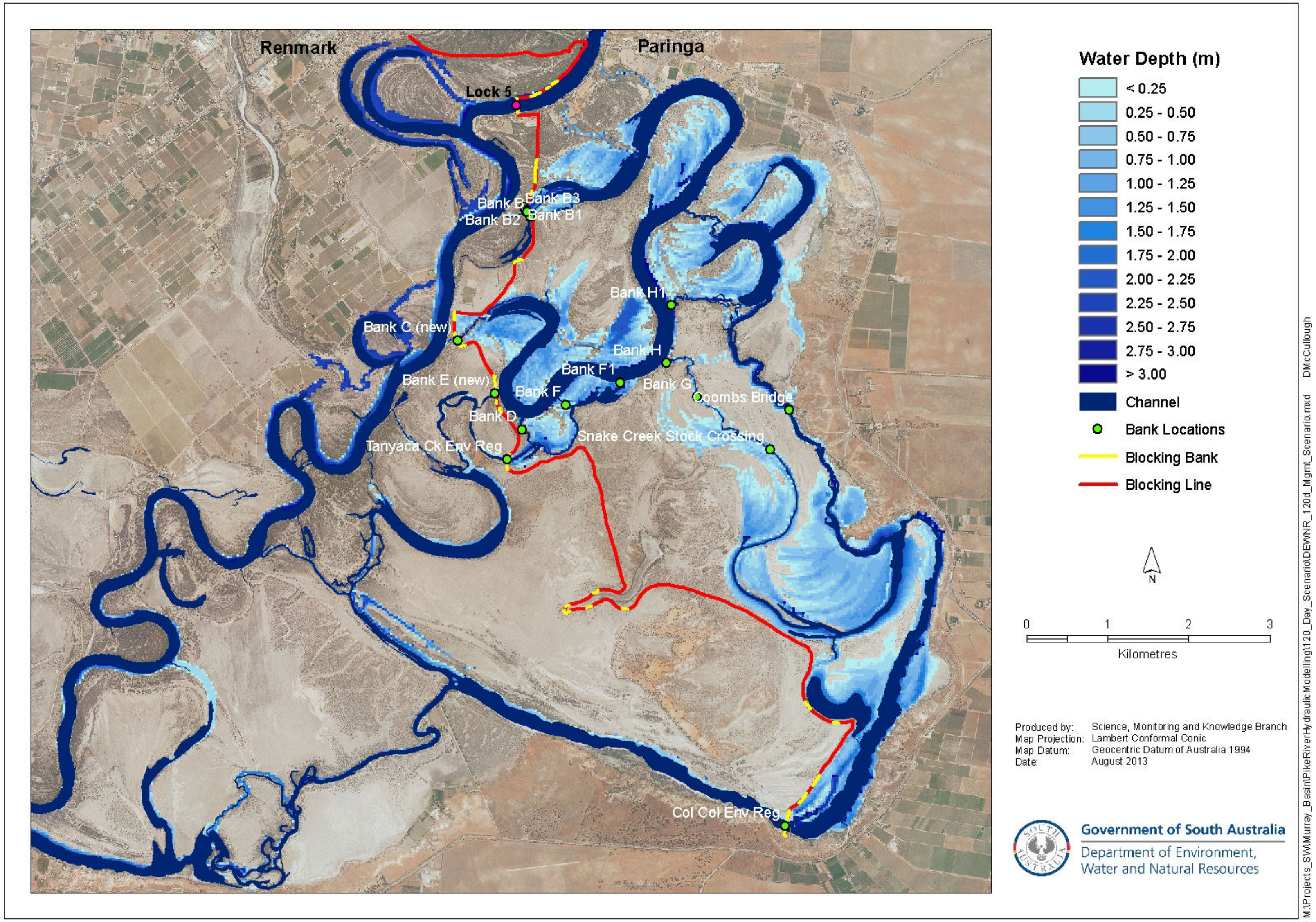

Figure 10: Inundation extent during Pike environmental watering, Day 20 of operation.

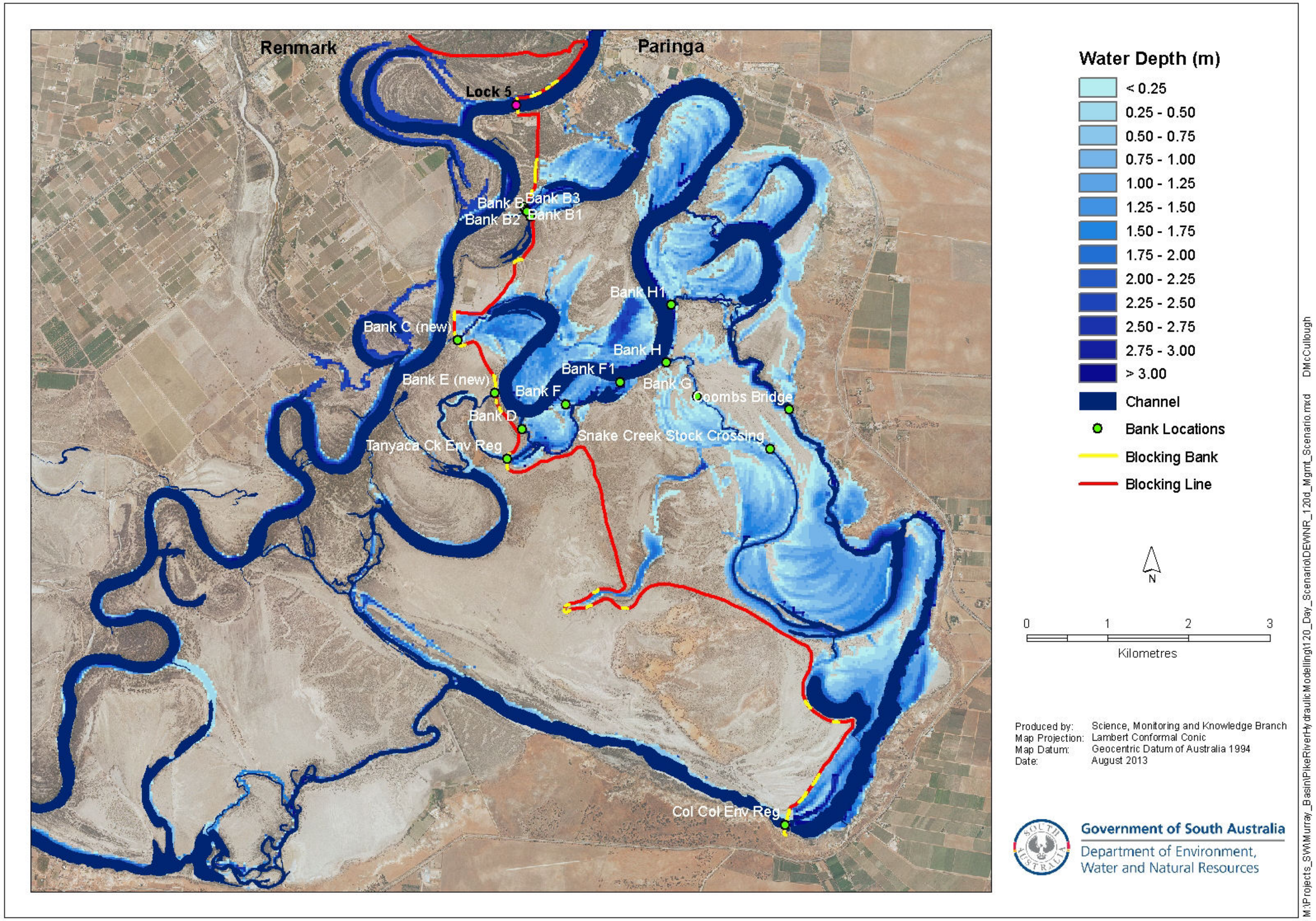

Figure 11: Inundation extent during Pike environmental watering, Day 30 of operation.

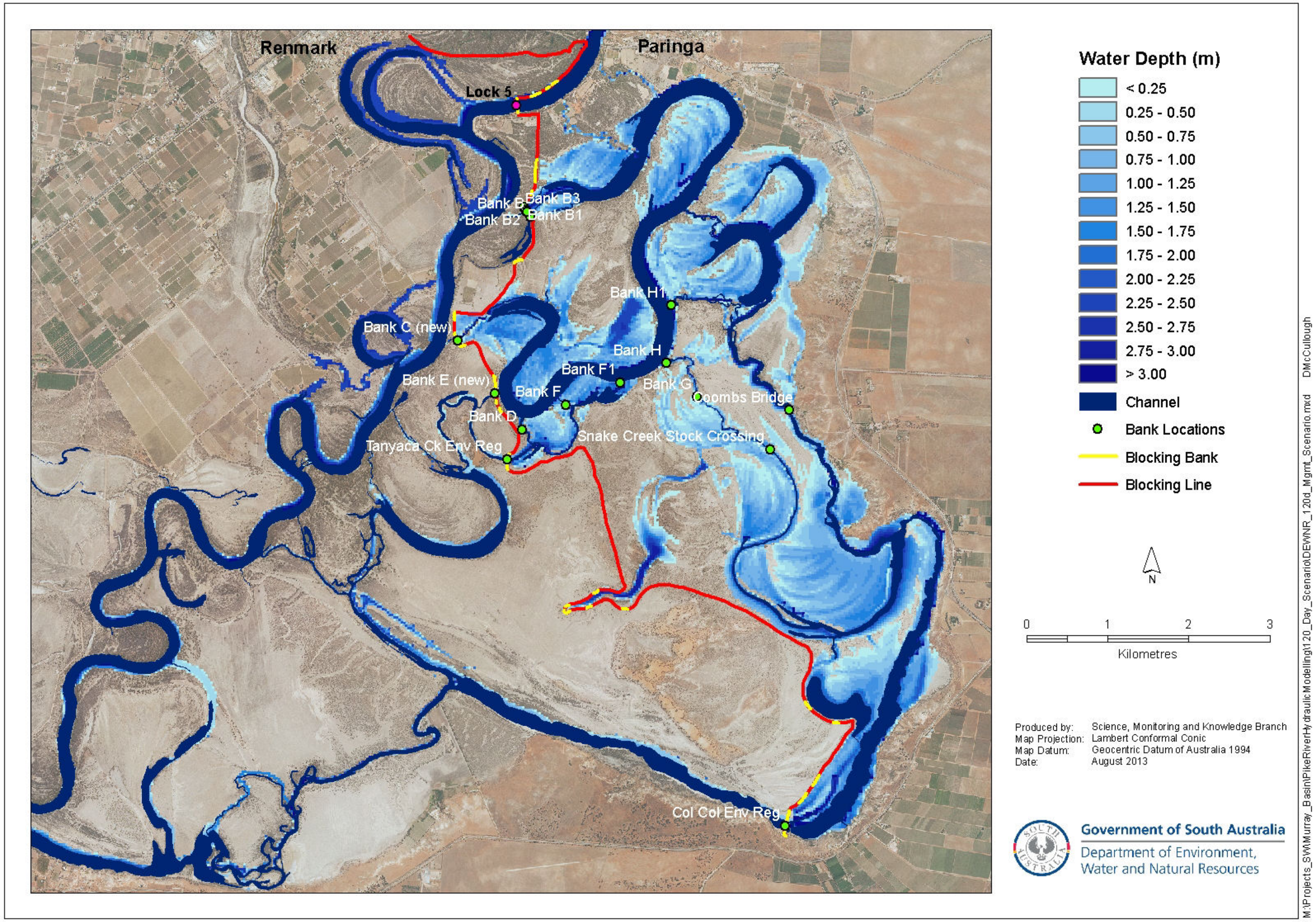

Figure 12: Inundation extent during Pike environmental watering, Day 40 of operation.

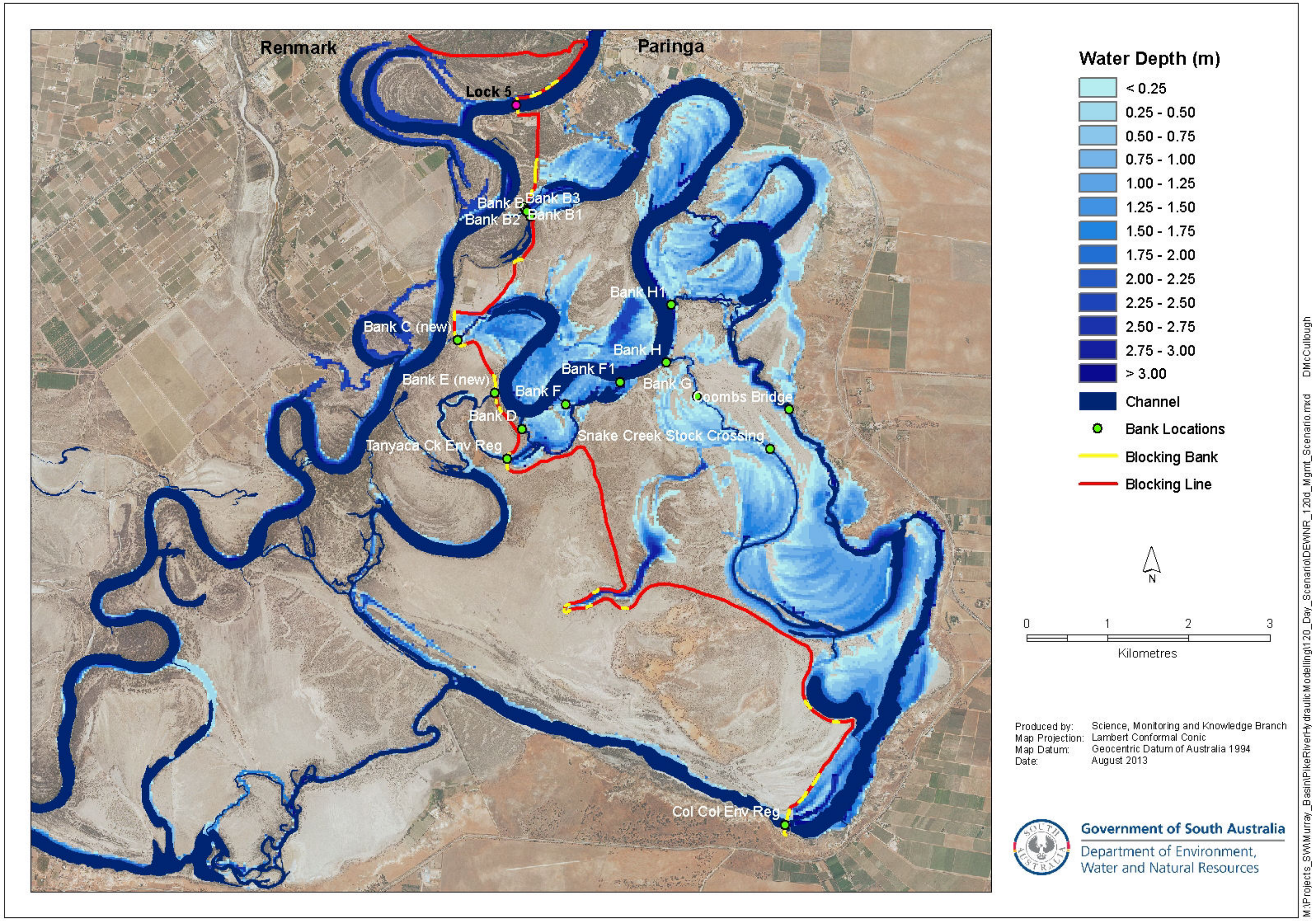

Figure 13: Inundation extent during Pike environmental watering, Day 50 of operation.

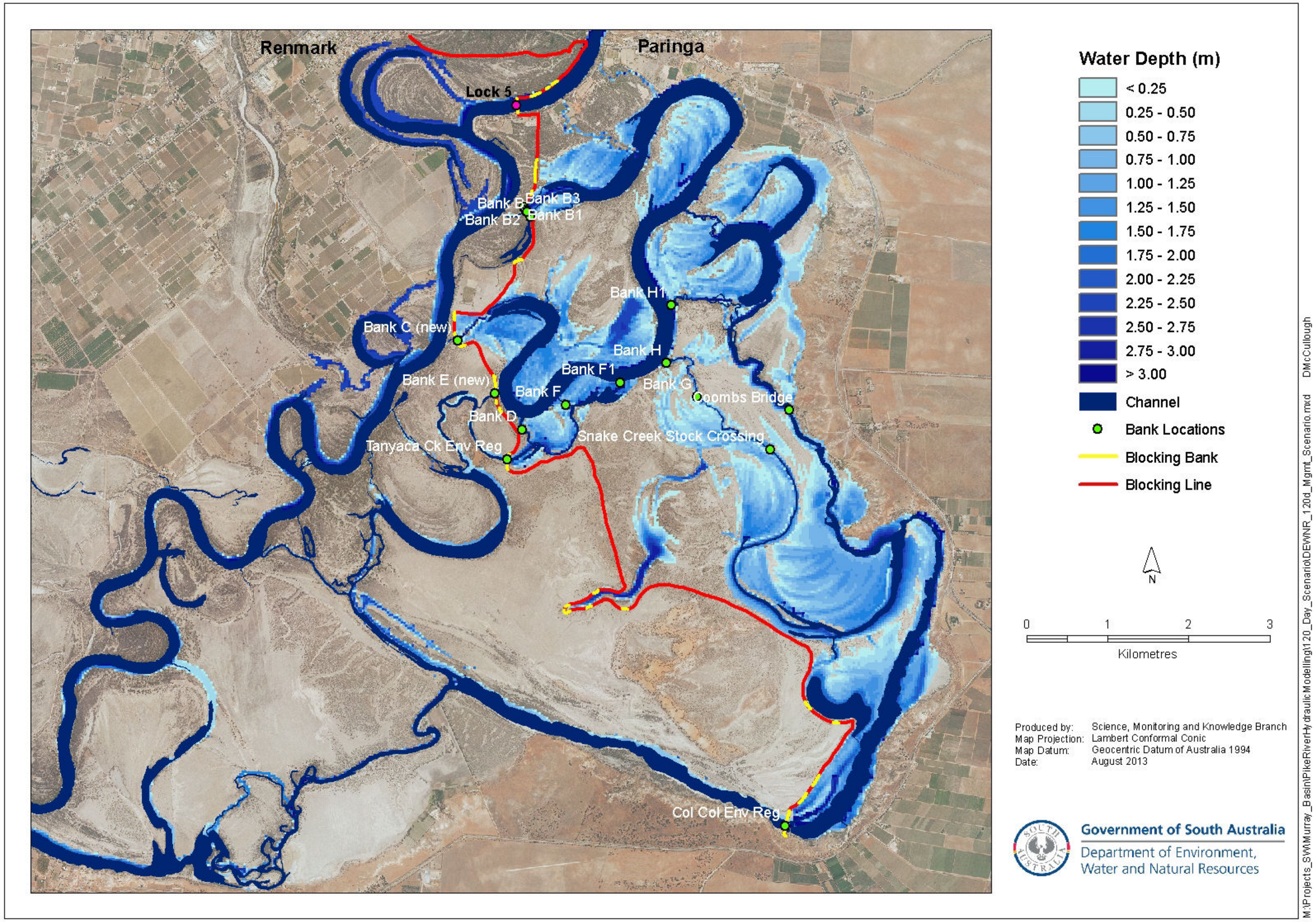

Figure 14: Inundation extent during Pike environmental watering, Day 60 of operation.

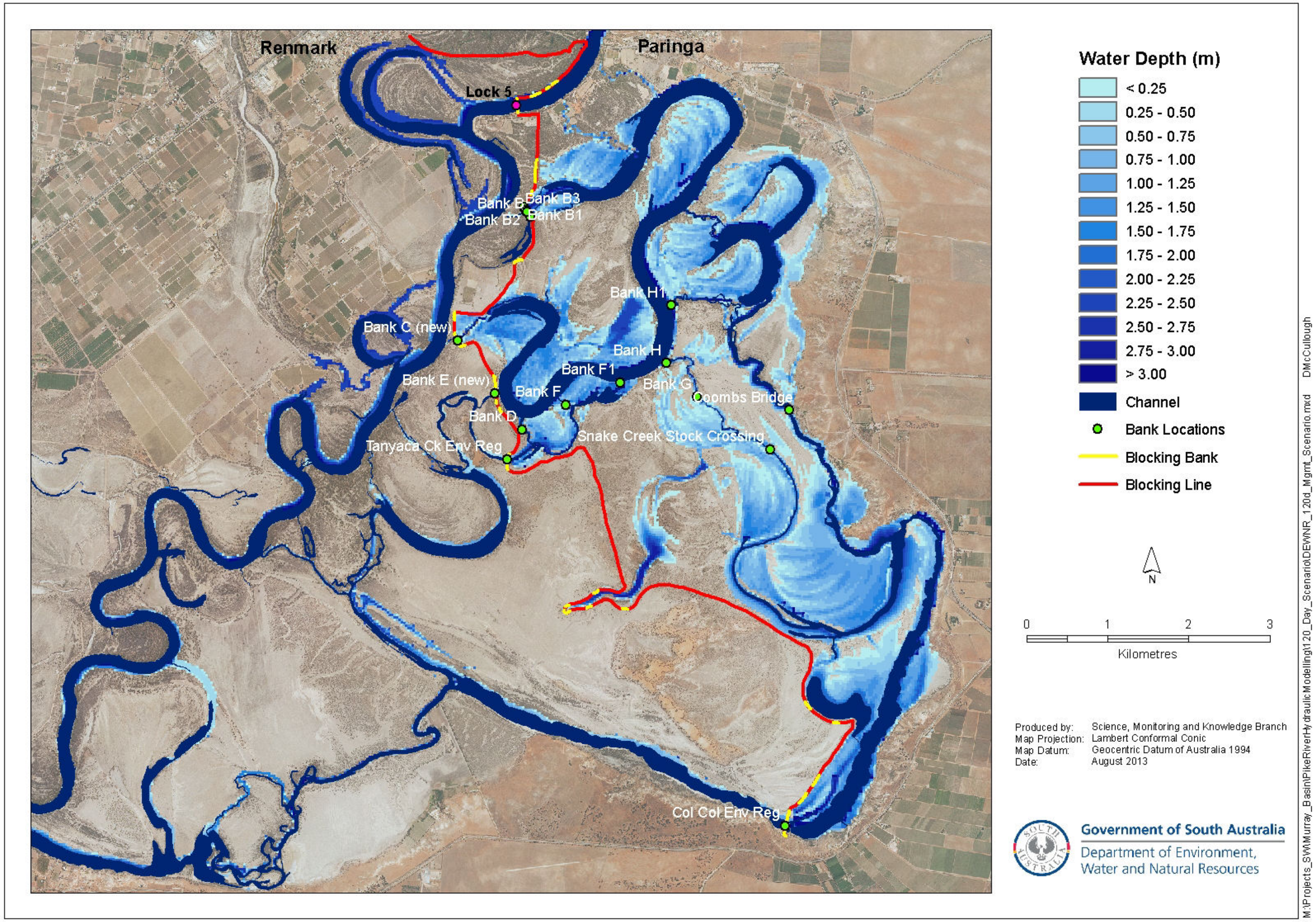

Figure 15: Inundation extent during Pike environmental watering, Day 70 of operation.

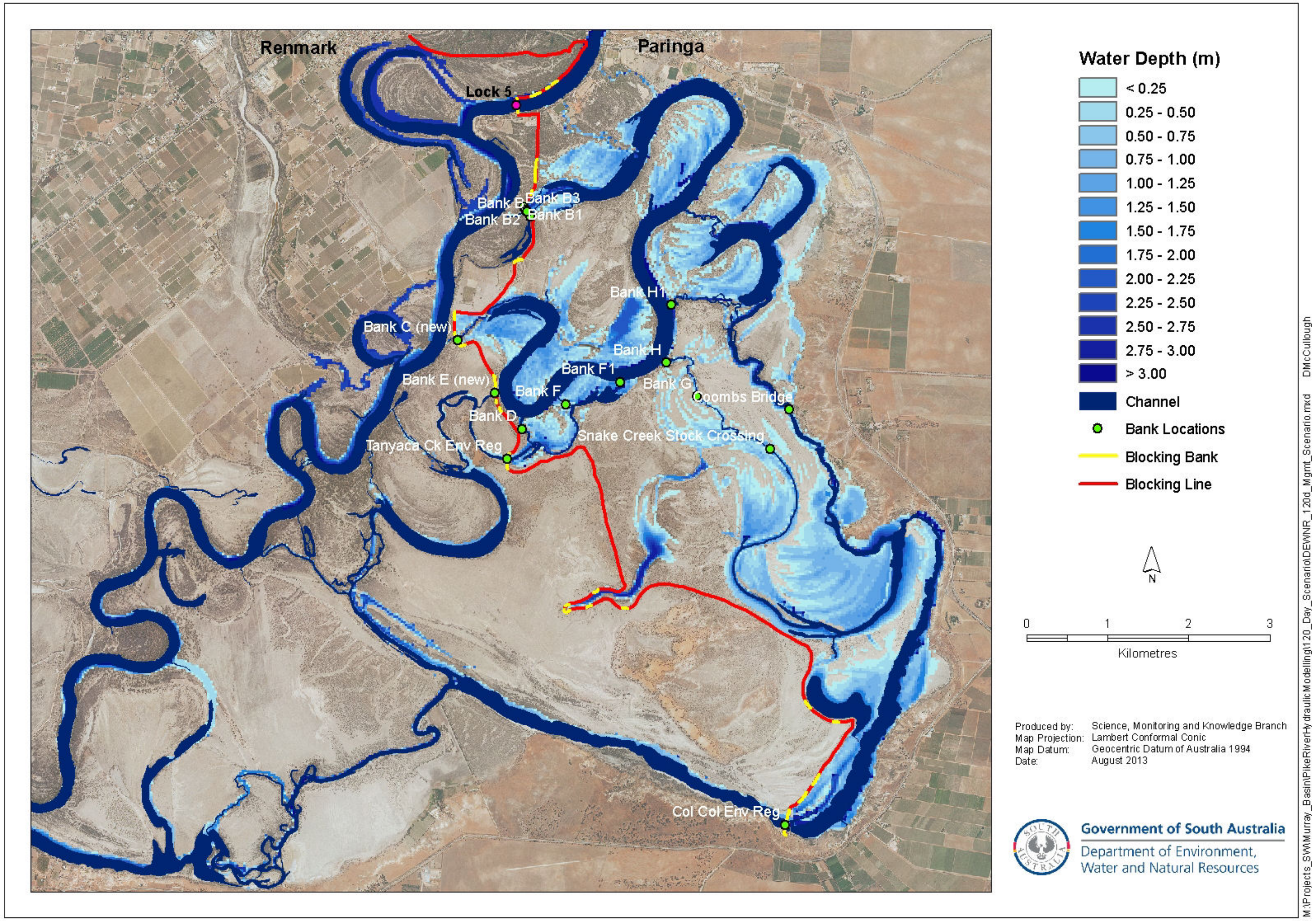

Figure 16: Inundation extent during Pike environmental watering, Day 80 of operation.

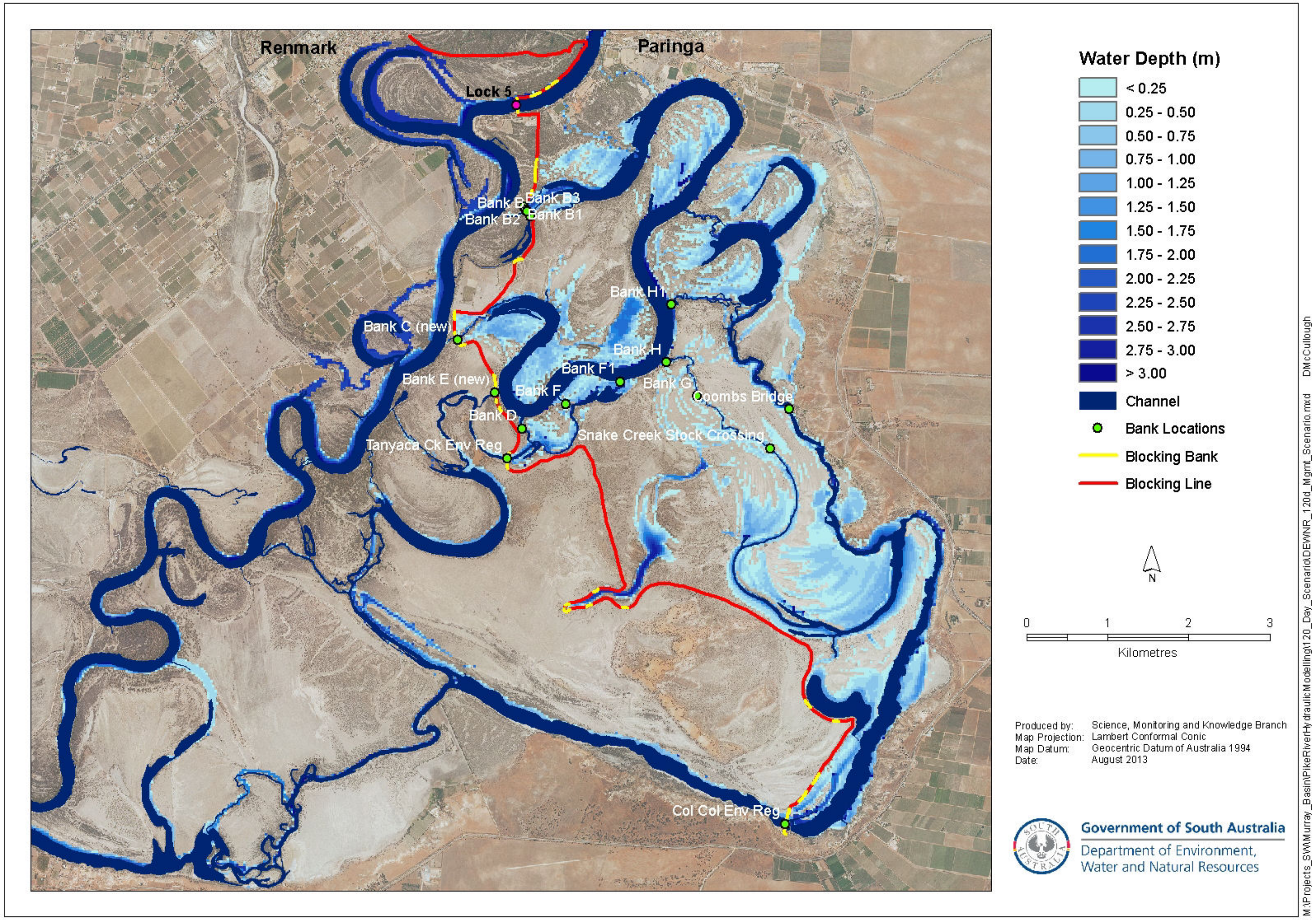

Figure 17: Inundation extent during Pike environmental watering, Day 90 of operation.

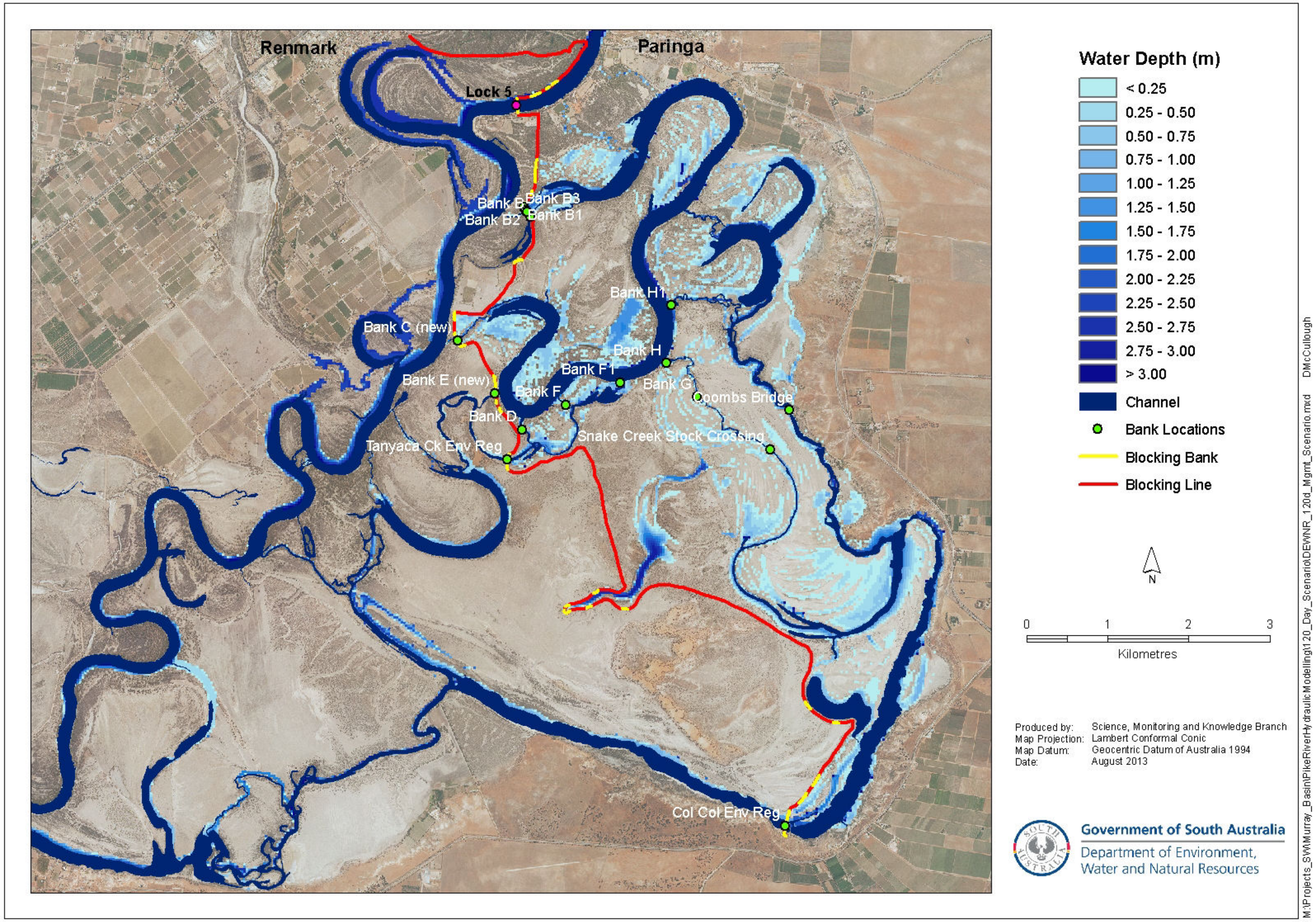

Figure 18: Inundation extent during Pike environmental watering, Day 100 of operation.

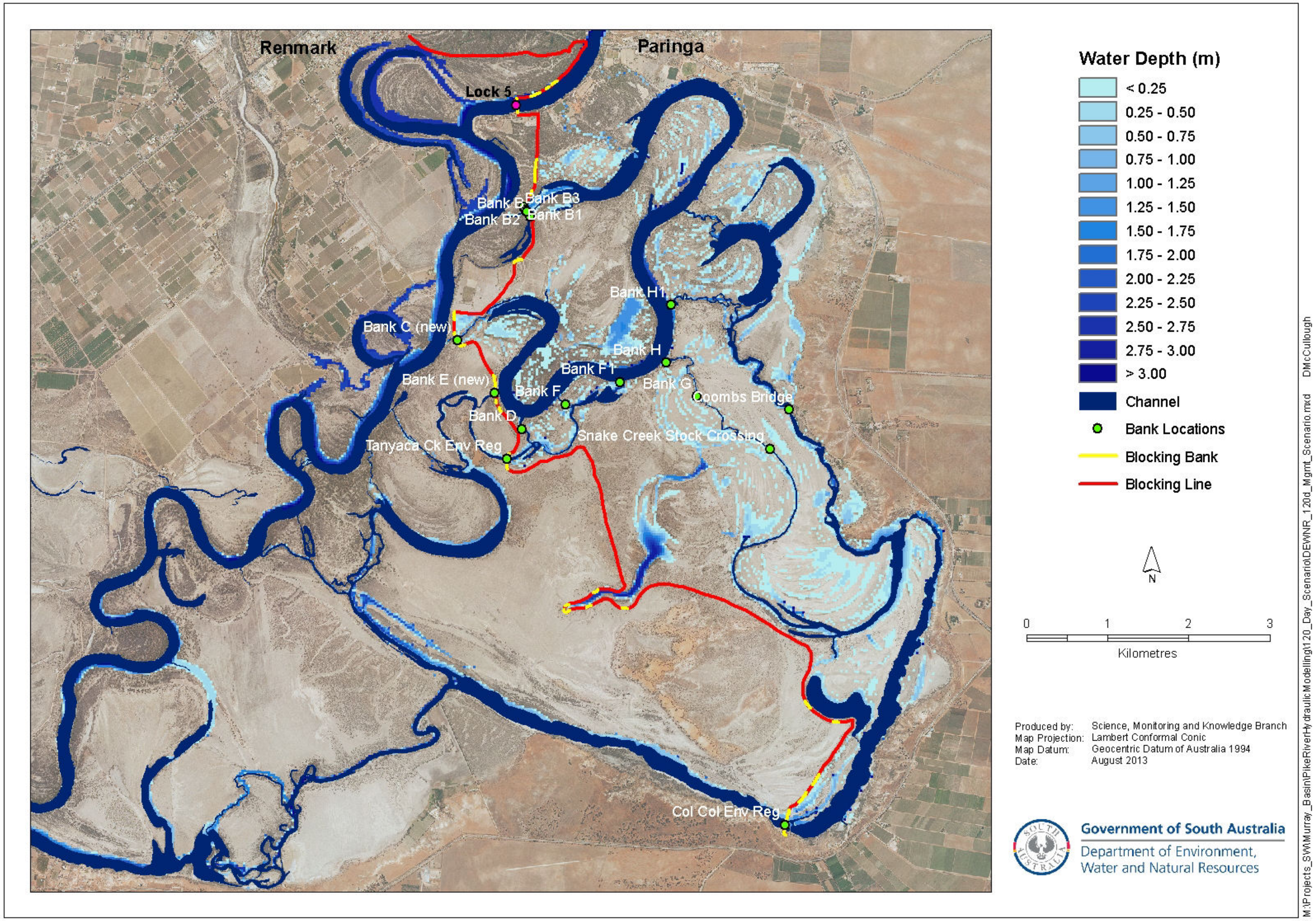

Figure 19: Inundation extent during Pike environmental watering, Day 110 of operation.

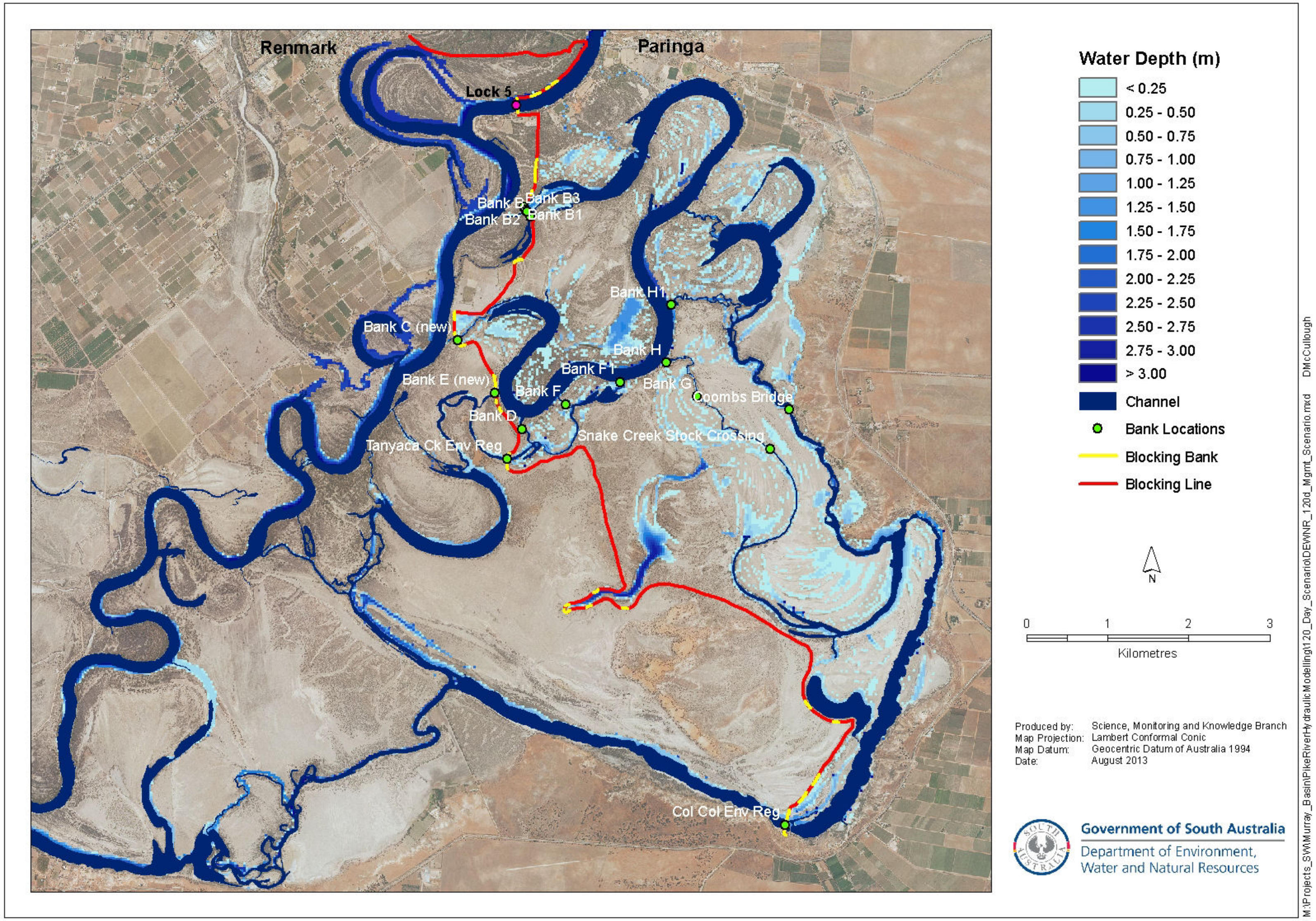

Figure 20: Inundation extent during Pike environmental watering, Day 120 of operation.

Table 6 shows water level, discharge, velocity and bed shear stress at Days 44, 54 (Holding Phase) and 100 (Draining Phase) for the current scenario at the reporting locations in Figure 1. Results from the previous Water Technology scenario at 30,000 ML/d flow at Lock 5 are also presented for comparison (N.B. only results from Day 44 of operation are reported in those results). Velocities highlighted in green indicate velocities ideal for fish passage. Additionally, velocity maps are shown for Day 44 (Tanyaca Q = 600 ML/d, Col Col Q = 400 ML/d; Figure 21), Day 54 (Tanyaca Q = 400 ML/d, Col Col Q = 600 ML/d; Figure 22) and Day 100 (Figure 23), with velocities conducive to fish passage shown by dark orange colouring. The results indicate that velocities immediately downstream of both Col Col and Tanyaca regulators are within the fish passage range of 0.18 to 1.4 m/s during both holding and draining phases, especially during the draining phase downstream of the Col Col regulator. Velocity in Rumpagunyah is also within the ideal range of fish passage during the draining phase.

Bed shear stress remains low at all reported locations relative to the risk of bank erosion, with the highest bed shear stress value being 1.3 N/m^2 in Tanyaca Creek upstream of Tanyaca lagoon (i.e. Location 11). A previous unpublished study on erosion impacts in the Chowilla anabranch system (which is located directly upstream of the Pike complex) indicates that no significant risk of bank erosion exists for clay soils at bed shear stresses of less than 11 N/m^2. Given that clay is the major component of soils within the main channels of the Pike system (ABARES, 2011), this suggests that operating with only Col Col and Tanyaca regulators as drainage points in the system does not adversely impact on erosion downstream of these regulators.

Despite there being little difference in operational conditions (other than flow at Lock 5) between the current and previous scenarios at Day 44, velocities and flows at certain locations upstream of the blocking bank differ markedly between the model outputs (note that differences downstream of the blocking bank are expected at different flow at Lock 5). In particular, velocities differ at Mundic Lagoon Outlet 1 (0.00 m/s current scenario to 0.10 m/s previous scenario); Mundic Lagoon Outlet 3 (0.01 m/s current to 0.19 m/s previous); Bank F1 (0.00 m/s current to 0.16 m/s previously); Pike River at Location 14 (0.00 m/s current to 0.25 m/s previously); and Northern Pike Lagoon (0.00 m/s current to 0.20 m/s previous). The reason for these differences is unclear, although the velocity map in Figure 21 suggests that velocities mainly fall within the 0.00 to 0.04 m/s range, and do not exceed approximately 0.1 m/s, upstream of the blocking bank (excluding Deep Creek and Margaret Dowling). It is also unlikely that, for instance, velocities would reach 0.20 m/s in Northern Pike Lagoon – which is in a backwater location with no flow-through – thereby supporting the low velocities reported at these locations in the current scenario.

4.3 Summary

The operational management scenario demonstrates that utilising Col Col and Tanyaca regulators as the sole drainage points in the system following managed inundation is feasible without requiring additional drainage capacity through Banks B and C. The lower rates of drainage results in the water level reducing at a slower rate than when Banks B and C are utilised for additional drainage, resulting in the drainage phase of operation spanning approximately 40 days (i.e. approximately 18 days were required for drainage in the previous scenario).

Rates of water level reduction are maintained well below the prescribed maximum rate of 10 cm/d, resulting in preferable conditions for avoiding bank slumping. Note however that additional drainage capacity is available through both Tanyaca and Col Col regulators, suggesting that drainage rates can be increased through these regulators without the need for further drainage capacity elsewhere if a shorter operating period is desired.

Table 6: Water level, discharge, velocity and bed shear stress at Days 44, 54 and 100 of operational management scenario. Results from previous scenario also included for comparison (N.B. only Day 44 reported on previously by Water Technology).

Reporting Location /Stream Name		Previous Scenario, Day 44 30,000 ML/d[1]				Current Scenario, Day 44 10,000 ML/d				Current Scenario, Day 54 10,000 ML/d				Current Scenario, Day 100 10,000 ML/d			
		h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²
1	Deep Creek	16.45	600	0.05	0.27	16.45	600	0.23	0.36	16.45	600	0.23	0.36	15.77	600	0.39	1.12
2	Margaret Dowling	16.41	400	0.08	0.18	16.41	400	0.11	0.08	16.41	400	0.11	0.08	15.48	400	0.21	0.32
3	Mundic Lagoon - Bank B	16.42	0	0.00	0.01	16.40	0.5	0.00	0.00	16.41	0.5	0.00	0.00	15.45	7	0.00	0.00
4	Mundic Lagoon	16.42	938	0.02	0.02	16.40	990	0.02	0.00	16.40	990	0.02	0.00	15.44	1028	0.06	0.01
5	Mundic Lagoon Outlet 1	0.00	0	0.10	0.00	16.40	0	0.00	0.00	16.40	0	0.00	0.00	15.44	0	0.00	0.00
6	Mundic Lagoon Outlet 2	16.40	32	0.04	0.00	16.40	79	0.01	0.00	16.40	120	0.02	0.00	15.43	449	0.09	0.05
7	Mundic Lagoon Outlet 3	16.40	27	0.19	0.00	16.40	37	0.01	0.00	16.40	84	0.01	0.00	15.43	535	0.14	0.14
8	Upper Pike River	16.42	224	0.06	0.00	16.40	224	0.00	0.00	16.40	319	0.01	0.00	15.42	563	0.02	0.00
9	Snake Creek - Bank G	16.40	90	0.02	0.00	16.40	89	0.02	0.00	16.40	129	0.02	0.01	15.43	103	0.05	0.05
10	Tanyaca Creek - Bank F1	16.42	145	0.16	0.04	16.40	185	0.00	0.00	16.40	125	0.00	0.00	15.43	29	0.00	0.00
11	Bank D	N/A	N/A	N/A	N/A	16.40	88	0.01	0.00	16.40	41	0.01	0.00	15.44	4	0.00	0.00
12	Mundic Lagoon - Bank C	16.42	3	0.01	0.01	16.40	0	0.00	0.00	16.40	0	0.00	0.00	15.44	1	0.00	0.00
13	Pike River	16.42	350	0.00	0.15	16.40	400	0.01	0.00	16.39	600	0.02	0.00	15.30	1370	0.19	0.17
14	Pike River	16.42	383	0.25	0.01	16.39	400	0.00	0.00	16.39	600	0.01	0.00	15.25	1427	0.04	0.00
15	Lower Pike River*	14.05	3746	0.02	0.33	13.39	564	0.04	0.01	13.39	587	0.04	0.01	13.45	853	0.06	0.02
16	Northern Pike Lagoon	16.42	6	0.20	0.00	16.40	4	0.00	0.00	16.40	6	0.00	0.00	15.42	8	0.00	0.00
17	Rumpagunyah*	N/A	N/A	N/A	N/A	13.39	437	0.13	0.12	13.39	413	0.12	0.11	13.41	647	0.19	0.25
18	Pike River Outlet*	N/A	N/A	N/A	N/A	13.31	479	0.04	0.01	13.31	496	0.04	0.01	13.32	710	0.06	0.01
19	Inlet U/S EC pontoon*	N/A	N/A	N/A	N/A	13.31	84	0.02	0.01	13.31	91	0.03	0.01	13.32	143	0.04	0.02
20	Swift Creek*	N/A	N/A	N/A	N/A	13.44	0	0.00	0.00	13.44	0	0.00	0.00	13.45	0	0.00	0.00
21	Wood Duck Creek*	N/A	N/A	N/A	N/A	13.43	0	0.00	0.00	13.43	0	0.00	0.00	13.45	0	0.00	0.00
22	Tanyaca Ck u/s of Tanyaca Lagoon*	14.14	601	0.19	0.32	13.67	600	0.37	1.34	13.59	400	0.30	0.93	13.43	50	0.06	0.05

[1]Reported in previous Water Technology modelling results (locations 11 and 17 to 21 were not reported in this reference). *Locations downstream of blocking bank.

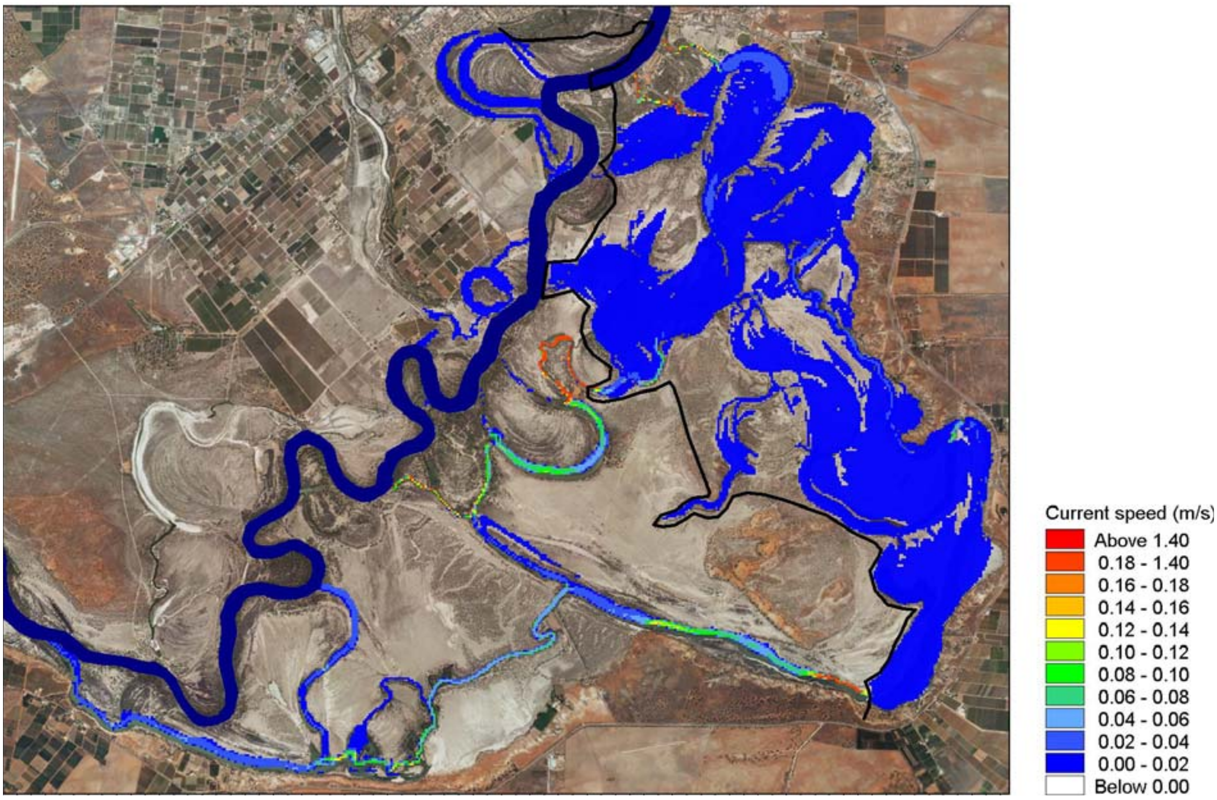

Figure 21: Velocity map at Day 44 of operational management scenario (during holding phase).

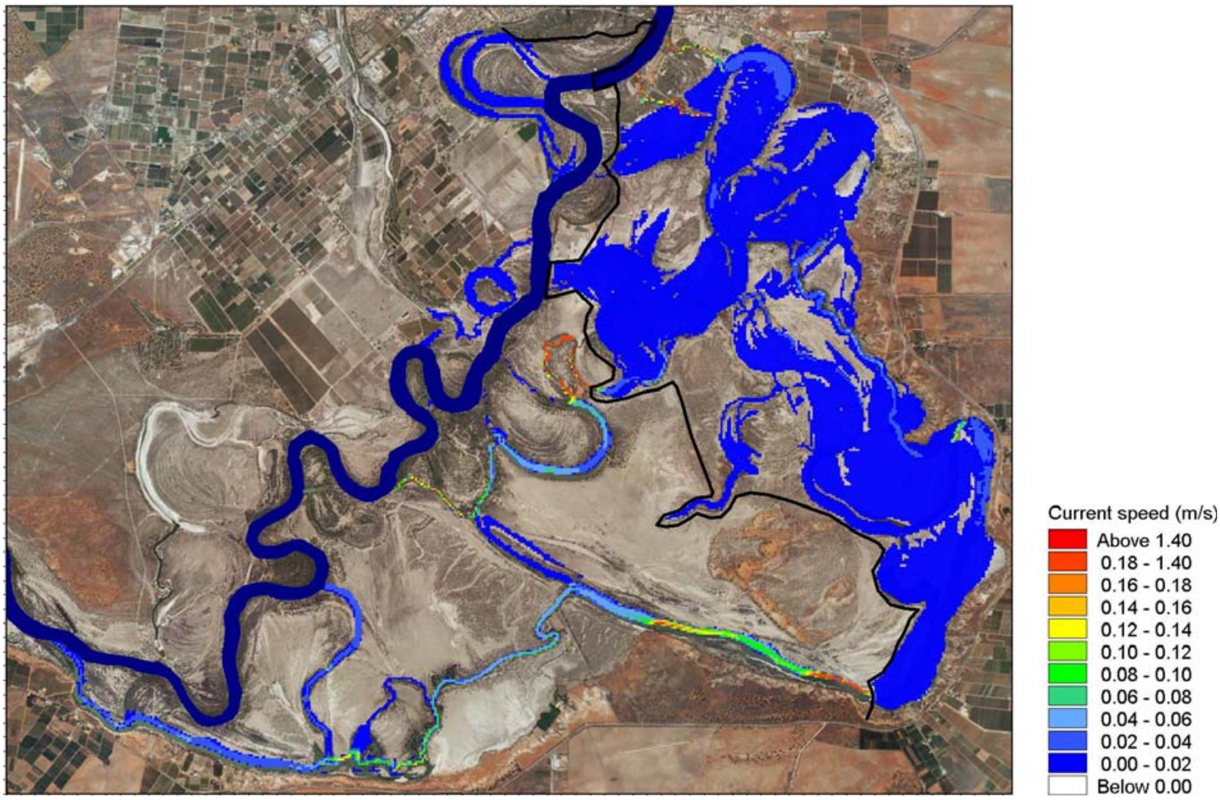

Figure 22: Velocity map at Day 54 of operational management scenario (during holding phase).

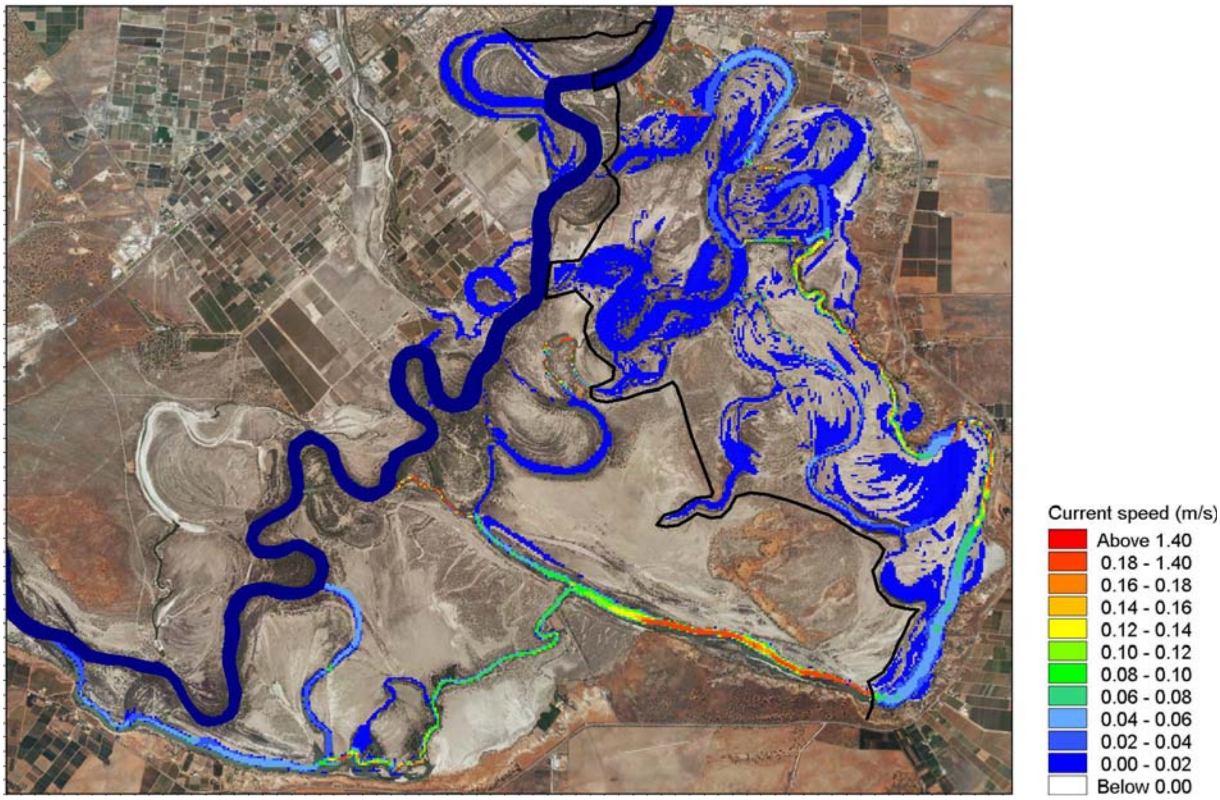

Figure 23: Velocity map at Day 100 of operational management scenario (during draining phase).

5. Scenario 3 – Managed Inundation with Staggered Filling Phase for Water Quality Maintenance

The following section presents the results of a modelling exercise to investigate the hydrological impacts on the Pike anabranch and associated floodplain of an operational management scheme, similar to that of Scenario 2, using a 90-day "staggered" filling phase to reduce the impact of hypoxic/anoxic blackwater generation on River Murray water quality.

5.1 Model Simulation

The model control scheme was designed to replicate a hypothetical hydrograph presented in previous unpublished reporting, which included a staggered rise and fall during the filling phase to promote a 20% exchange of water, as in Figure 24. Note that the hydrograph is designed to commence in June (instead of September as used in previous scenarios) to take advantage of lower water temperatures, and hence reduce the risk of low dissolved oxygen levels occurring during regulator operation.

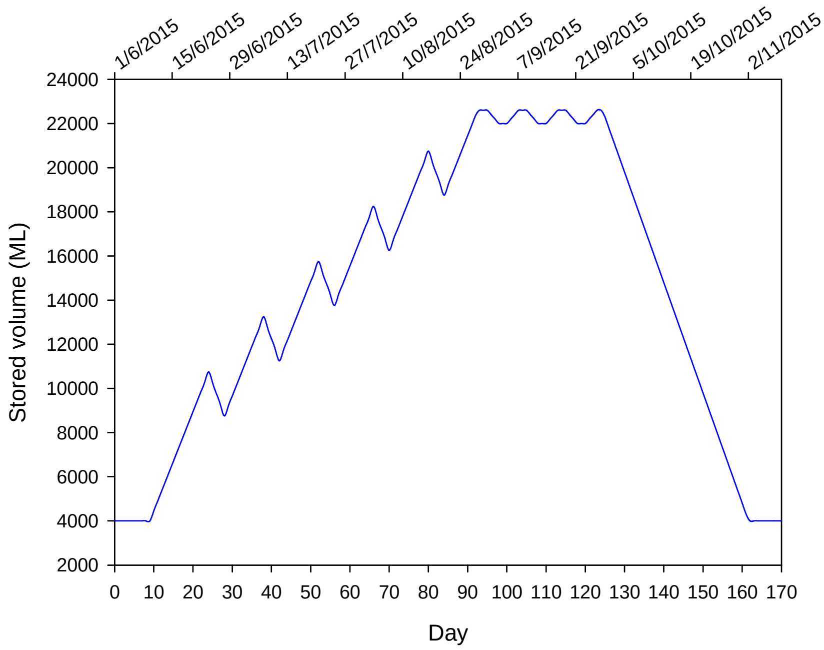

Figure 24: Suggested staggered rise and fall of water volume upstream of the Blocking Bank for dilution water containing low dissolved oxygen concentrations (unpublished report).

The following describes the model operational scheme, as illustrated in Figure 25:

- Lock 5 flow of 10,000 ML/d and an upstream water level of 16.8 m AHD for the 160 day period.
- A total of 1,000 ML/d flowing into the Pike system at Deep Creek (600 ML/d) and Margaret Dowling (400 ML/d) for 160 days.
- The Bank B complex and Bank C remain closed during operation to negate the requirement for fishway structures at these locations.

- **Filling phase:** Flows over the environmental regulating structures are varied throughout the filling phase to promote an exchange of approximately 20% of water behind the Blocking Bank. Combined flows over Tanyaca and Col Col regulators required to replicate the hydrograph in Figure 24 are 575 ML/d during each staggered filling (i.e. 525 ML/d over Col Col, 50 ML/d over Tanyaca) and 1,350 ML/d during each staggered draining (i.e. 1,000 ML/d at Col Col, 350 ML/d at Tanyaca). Water level upstream of Tanyaca Regulator is raised to 16.4 m AHD over approximately 90 days.
- **Holding phase:** Impounded water levels are held for approximately 4 weeks. During this phase, normal variations in flood levels are replicated by varying flows over each environmental regulator to a total flow of 1,000 ML/d (e.g. 600 ML/d over Col Col and 400 ML/d over Tanyaca Creek regulators, and vice versa).
- **Draining phase:** Water level is lowered at a rate of less than 10 cm/d by allowing 50 ML/d over Tanyaca and 1400 ML/d over Col Col regulators (note that maximum concept design flows through Tanyaca and Col Col regulators are 1300 ML/d and 3800 ML/d, respectively. Approximately 6 weeks are required to draw down the level in Mundic Lagoon upstream of the Blocking Bank to 'normal' operating levels (i.e. approximately 2 m reduction in head from full inundation extent).

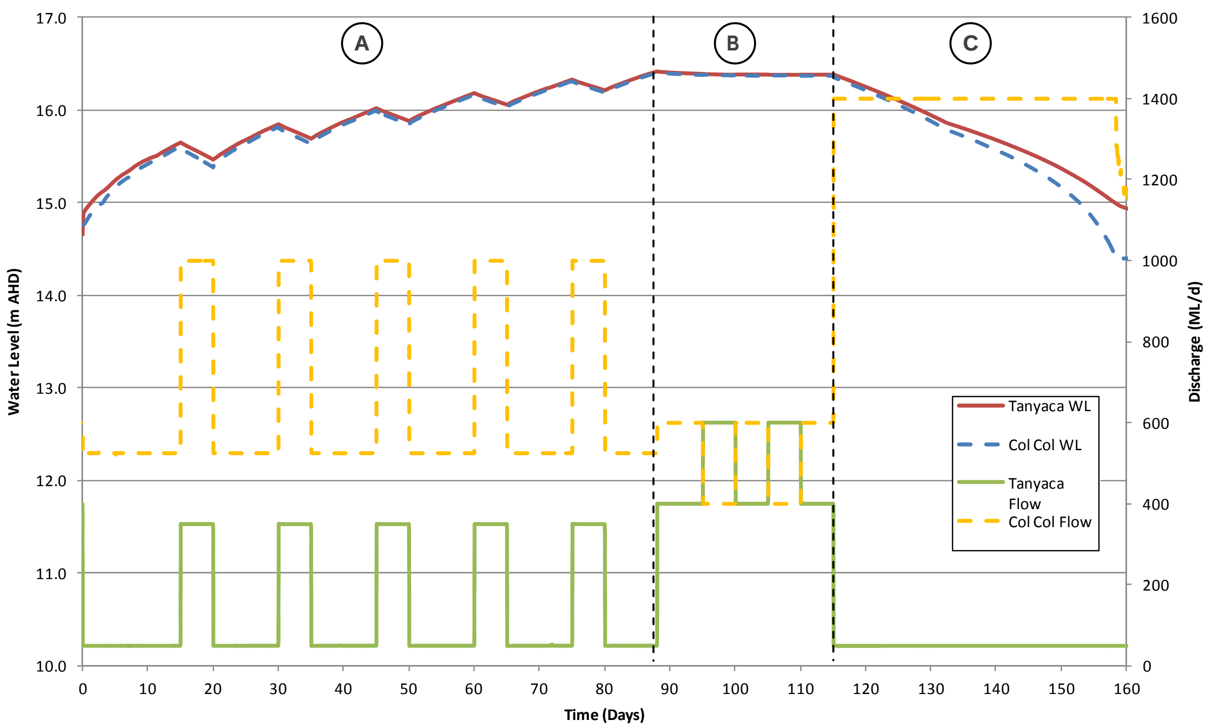

Figure 25: Water level and discharge at Tanyaca and Col Col regulators during operation. A – Filling Phase, B – Holding Phase, C – Draining Phase.

5.2 Results

Figure 26 shows the progress of inundated area, volume, and loss volume throughout the operational management scenario. Note that the maximum inundated area (approximately 1,680 ha) and maximum inundated volume (22,070 ML) behind the blocking bank are marginally reduced in comparison to the values calculated in Scenario 2 (1,700 ha and 22,400 ML, respectively; refer to Figure 6). These results amount to differences of only 1.1% in inundated area and 1.5% in inundated volume from the previous scenario, and as such are considered to be within the limits of model uncertainties.

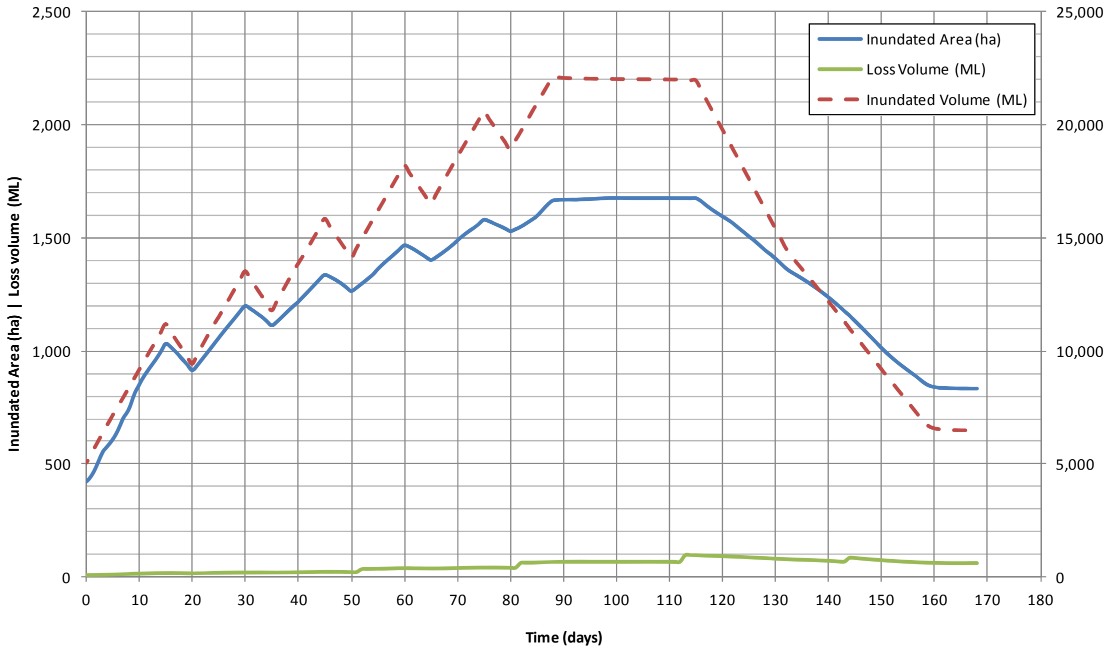

Figure 26: Area and volume inundated upstream of blocking bank for management scenarios, including volume of losses.

Total water balances for the scenario (with results from previous scenarios shown for comparison) are indicated in Table 7. Note that only volumes from start of the filling phase to end of the draining phase are considered to allow for comparison with previous scenario results. Total losses (7% of inflow volume) are reduced in comparison to losses from Scenario 2 (11% of inflow), which is a result of lower evaporation rates acting on the system due to operation commencing in June instead of September, as well as shorter period in the holding phase corresponding to the largest surface area. Correspondingly, outflow quantities are increased (93%) in comparison to Scenario 2 (89%). Storage remaining on the floodplain post-inundation is equivalent between Scenarios 2 and 3.

Table 7: Comparison of total water balances upstream of blocking alignment from start of filling phase to end of draining phase. Bracketed values indicate percentage of total inflow volume.

Water Balance Component (from start of filling phase to end of draining phase)	Total Volume – Existing Conditions[1] ML (% of inflow)	Total Volume – Scenario 2 ML (% of inflow)	Total Volume – Staggered Filling Phase ML (% of inflow)
Inflow	85,000	110,000	160,000
Outflow	78,900 (92.8%)	97,600 (88.7%)	149,250 (93.3%)
Losses	6,100 (7.2%)	9,700 (8.8%)	8,150 (5.0%)
Storage Change (final – initial volume)	0	2,700	2,600
Total Loss	6,100 (7.2%)	12,400 (11.3%)	10,750 (6.7%)

[1] Values obtained from Water Technology – results are identical regardless of Lock 5 flow assigned.

Figure 27 through to Figure 41 present the dynamic progress of inundation throughout the current scenario, from initial conditions to 160 days of operation. Each inundation map contains an inset hydrograph which indicates the represented stage of operation. Maximum inundation extent is represented by maps of Day 88 (Figure 37) and Day 115 (Figure 38).

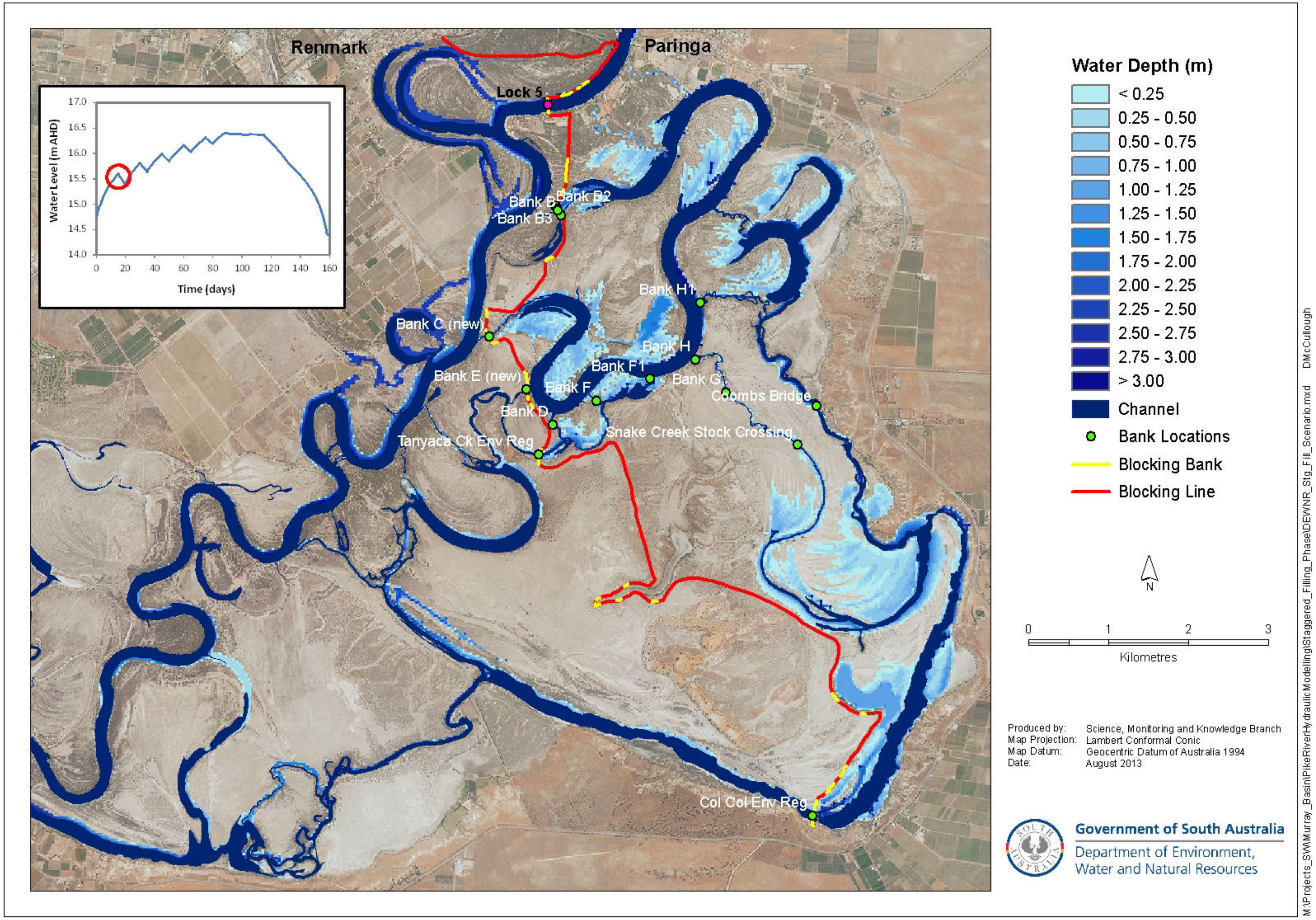

Figure 27: Inundation extent during staggered fill and drain, Day 15 of operation.

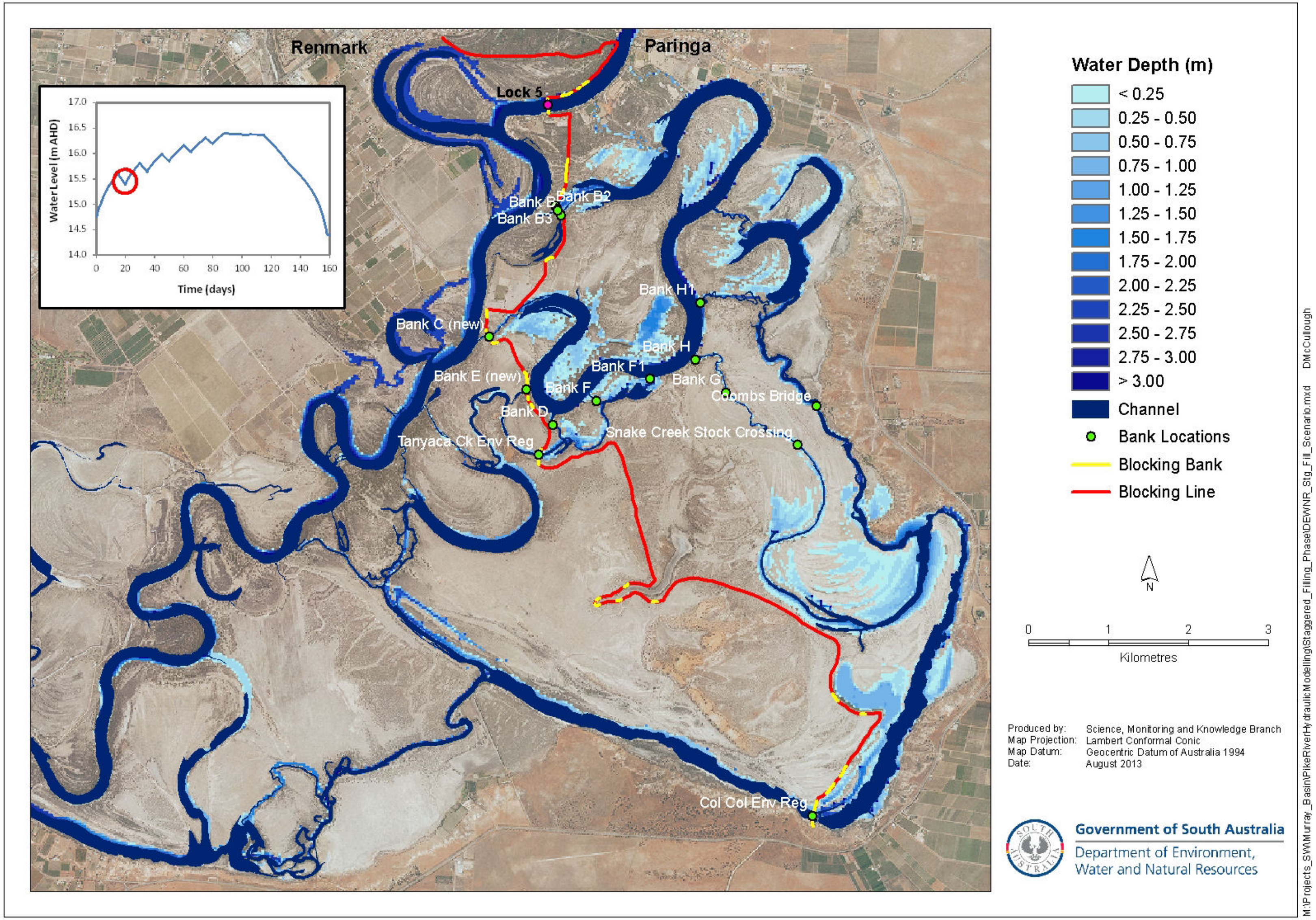

Figure 28: Inundation extent during staggered fill and drain, Day 20 of operation.

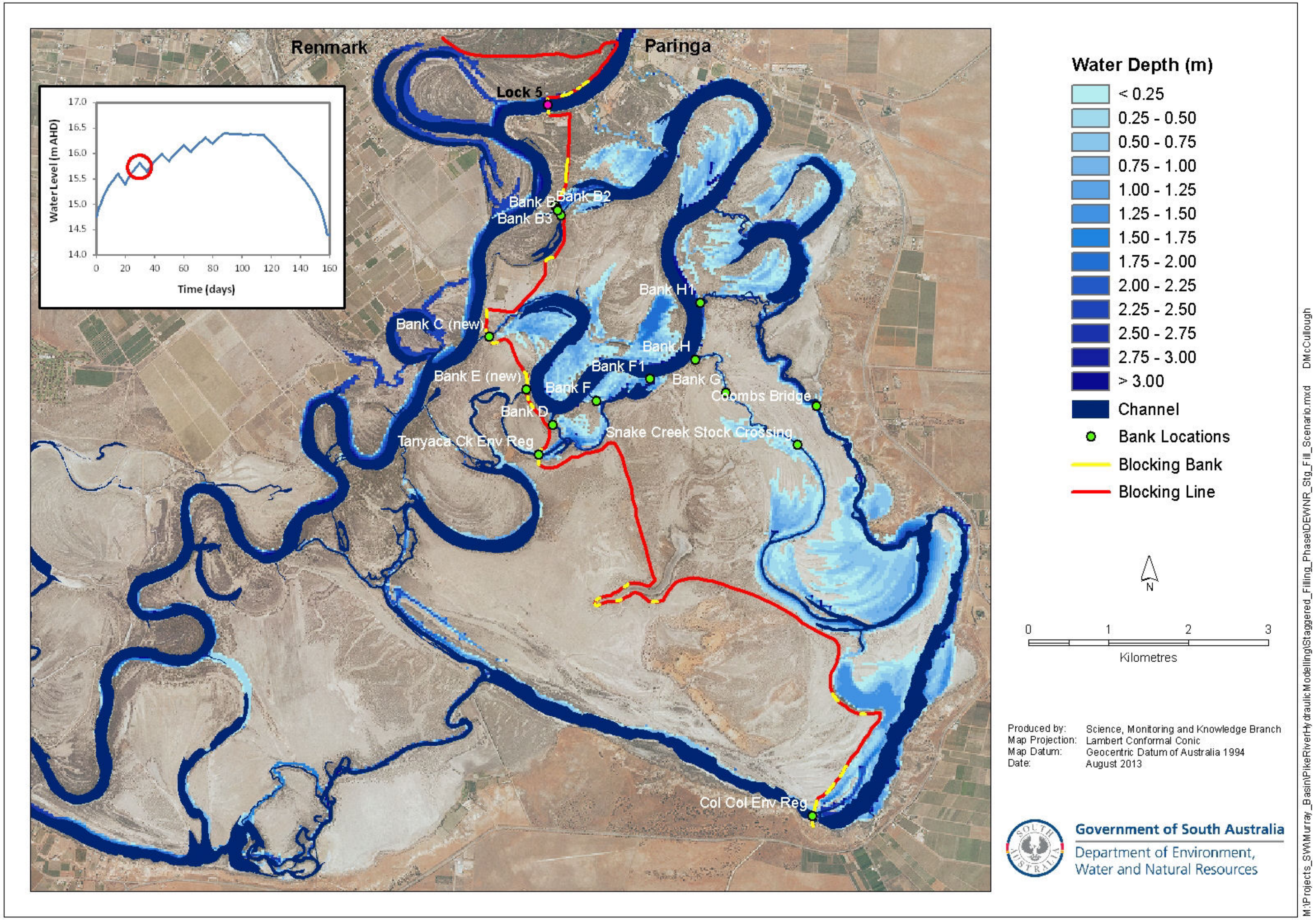

Figure 29: Inundation extent during staggered fill and drain, Day 30 of operation.

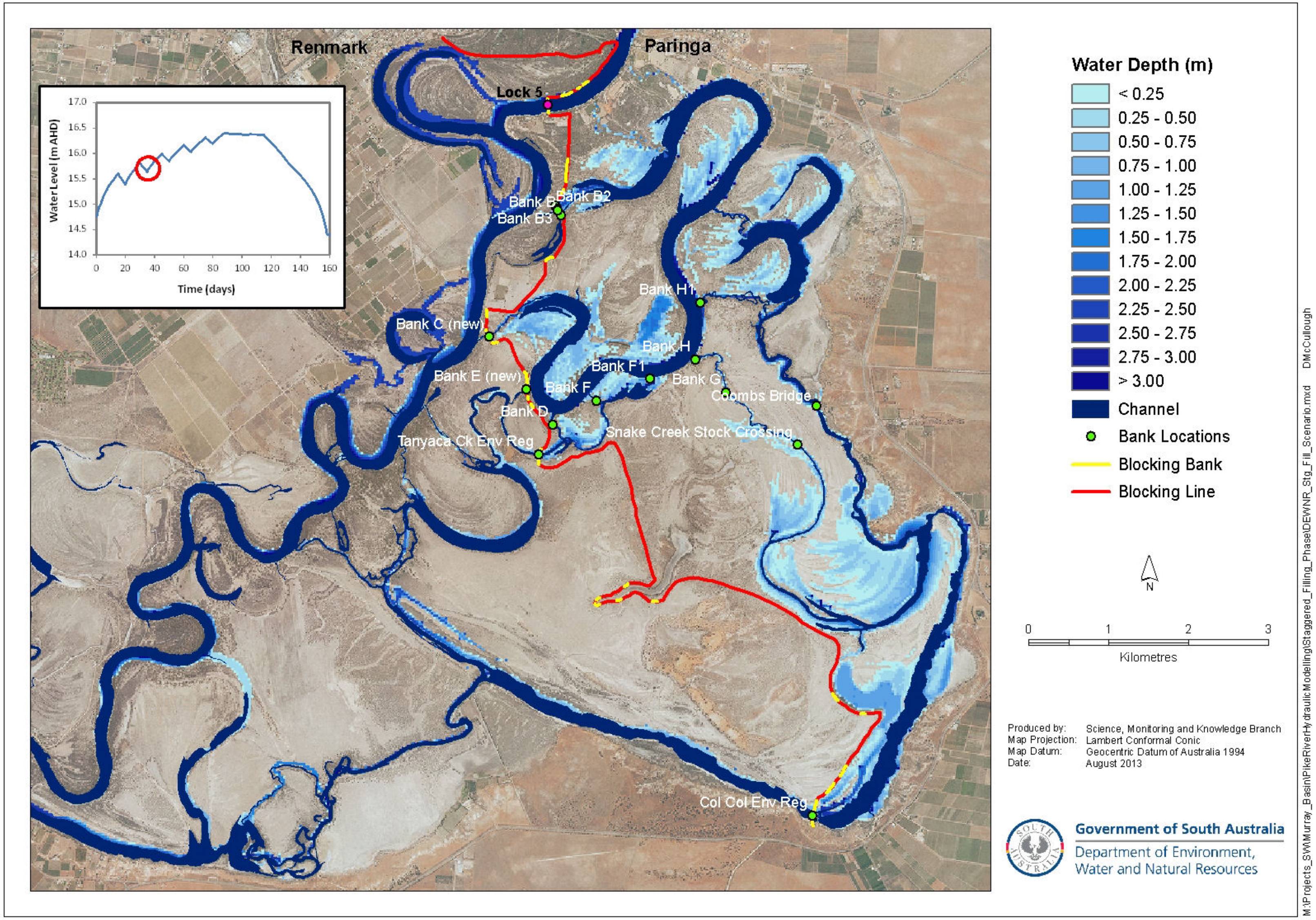

Figure 30: Inundation extent during staggered fill and drain, Day 35 of operation.

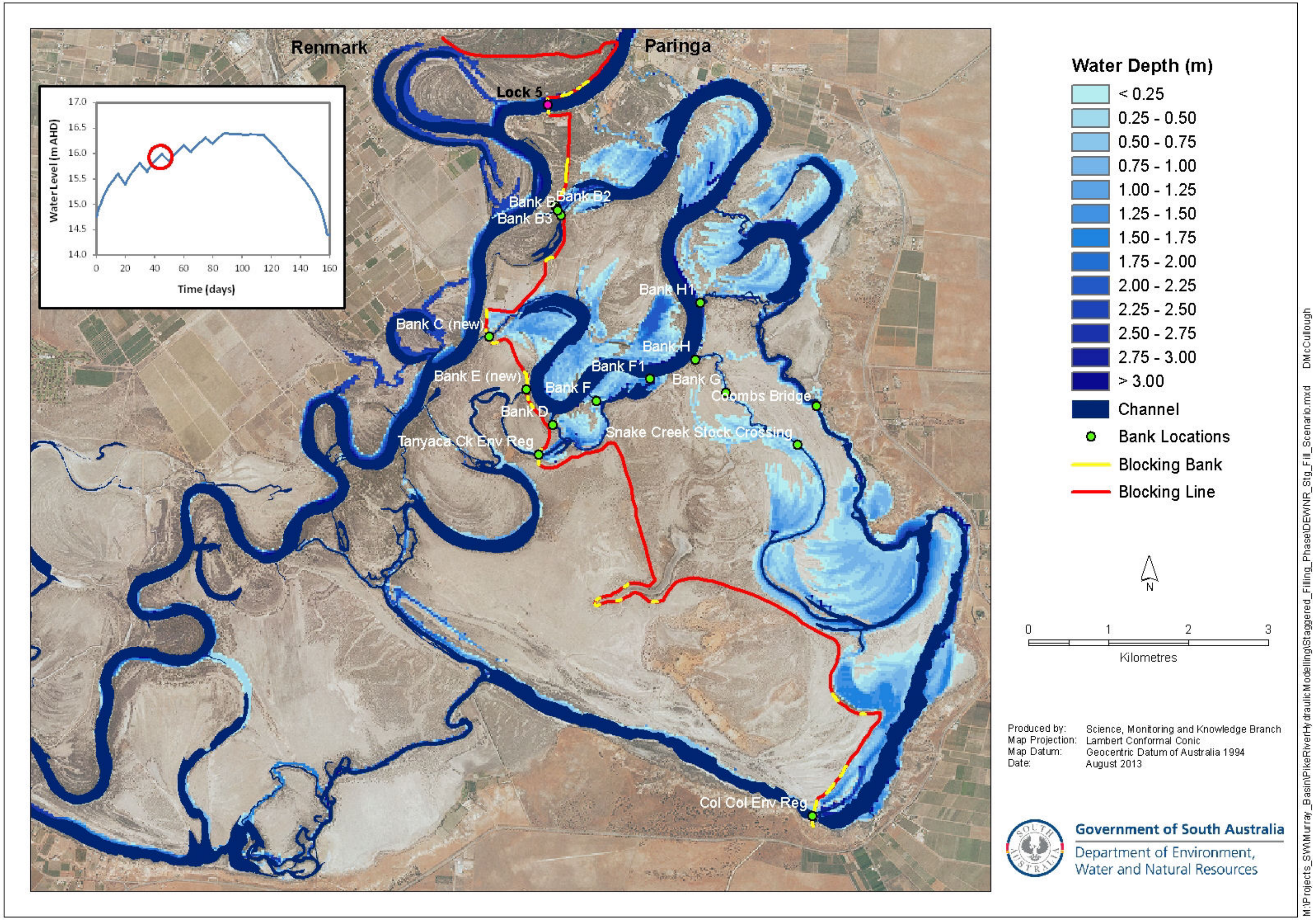

Figure 31: Inundation extent during staggered fill and drain, Day 45 of operation.

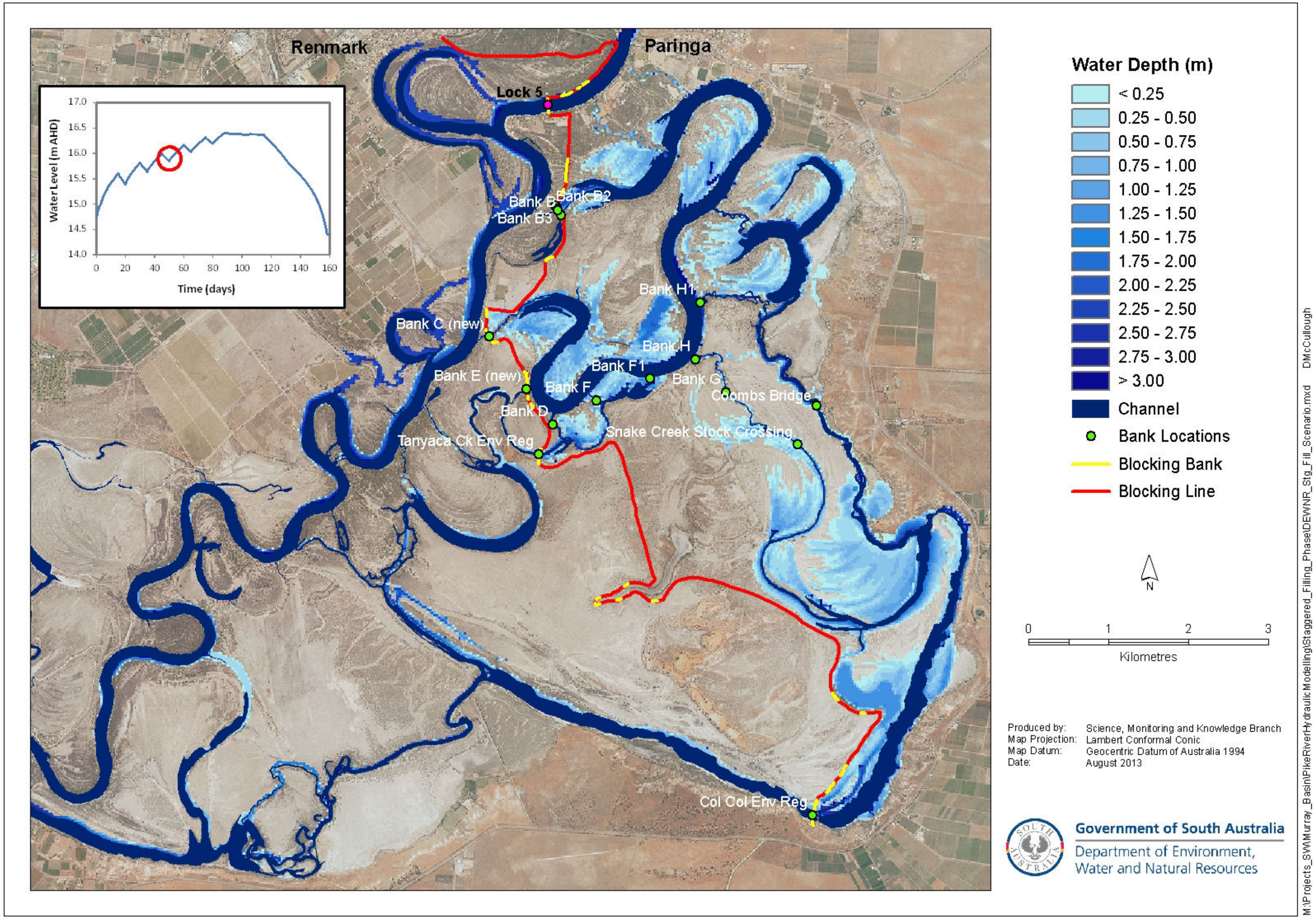

Figure 32: Inundation extent during staggered fill and drain, Day 50 of operation.

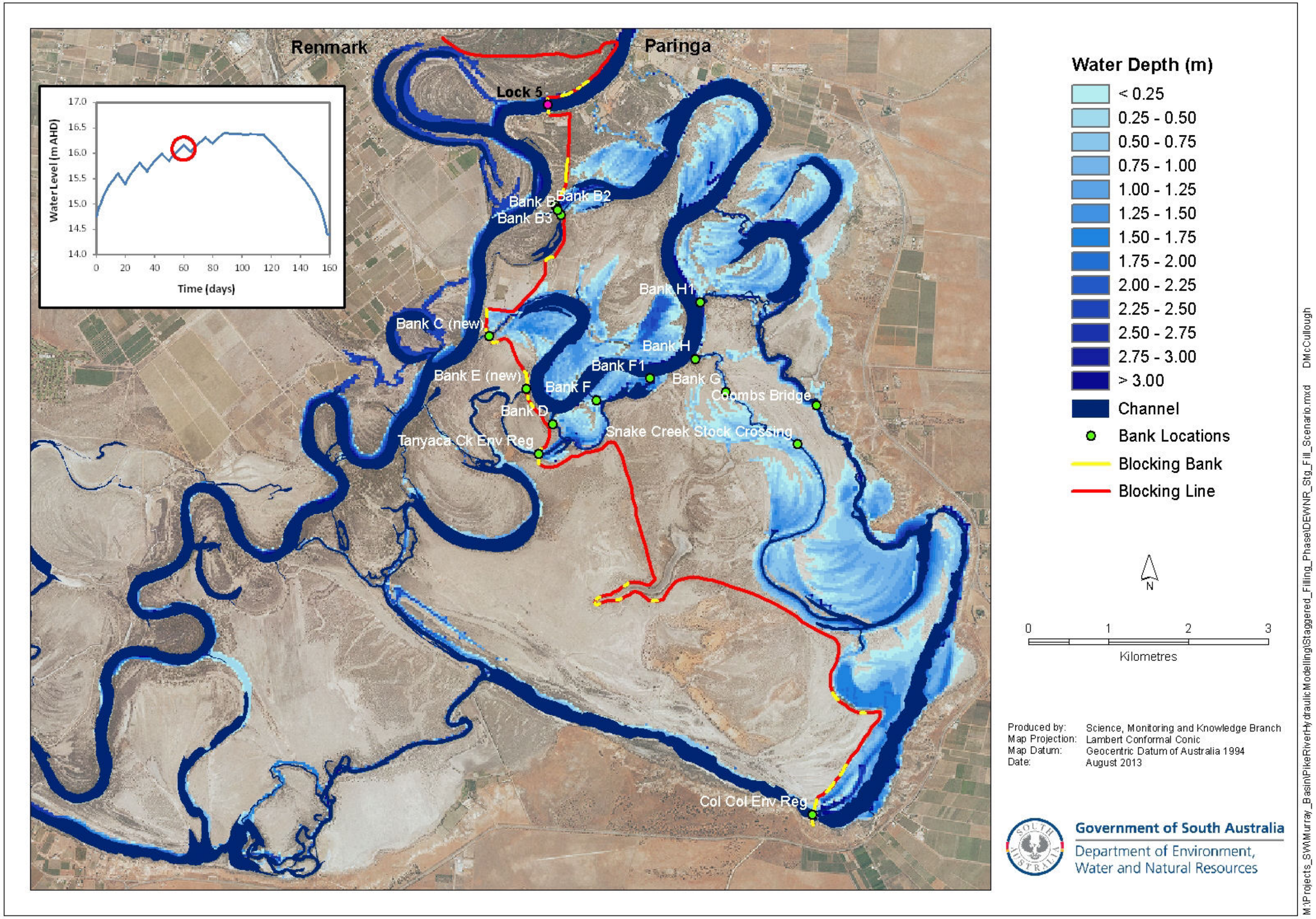

Figure 33: Inundation extent during staggered fill and drain, Day 60 of operation.

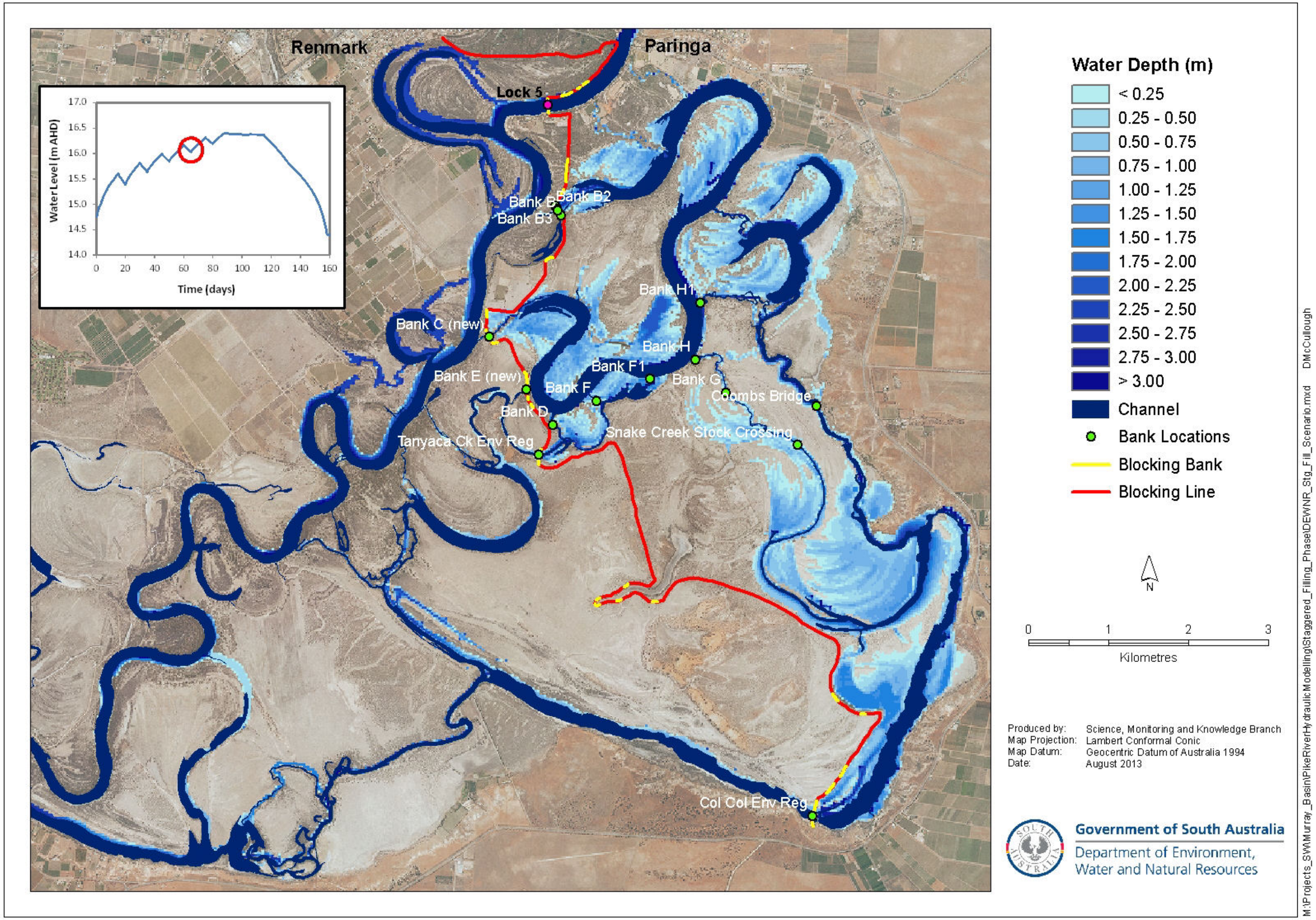

Figure 34: Inundation extent during staggered fill and drain, Day 65 of operation.

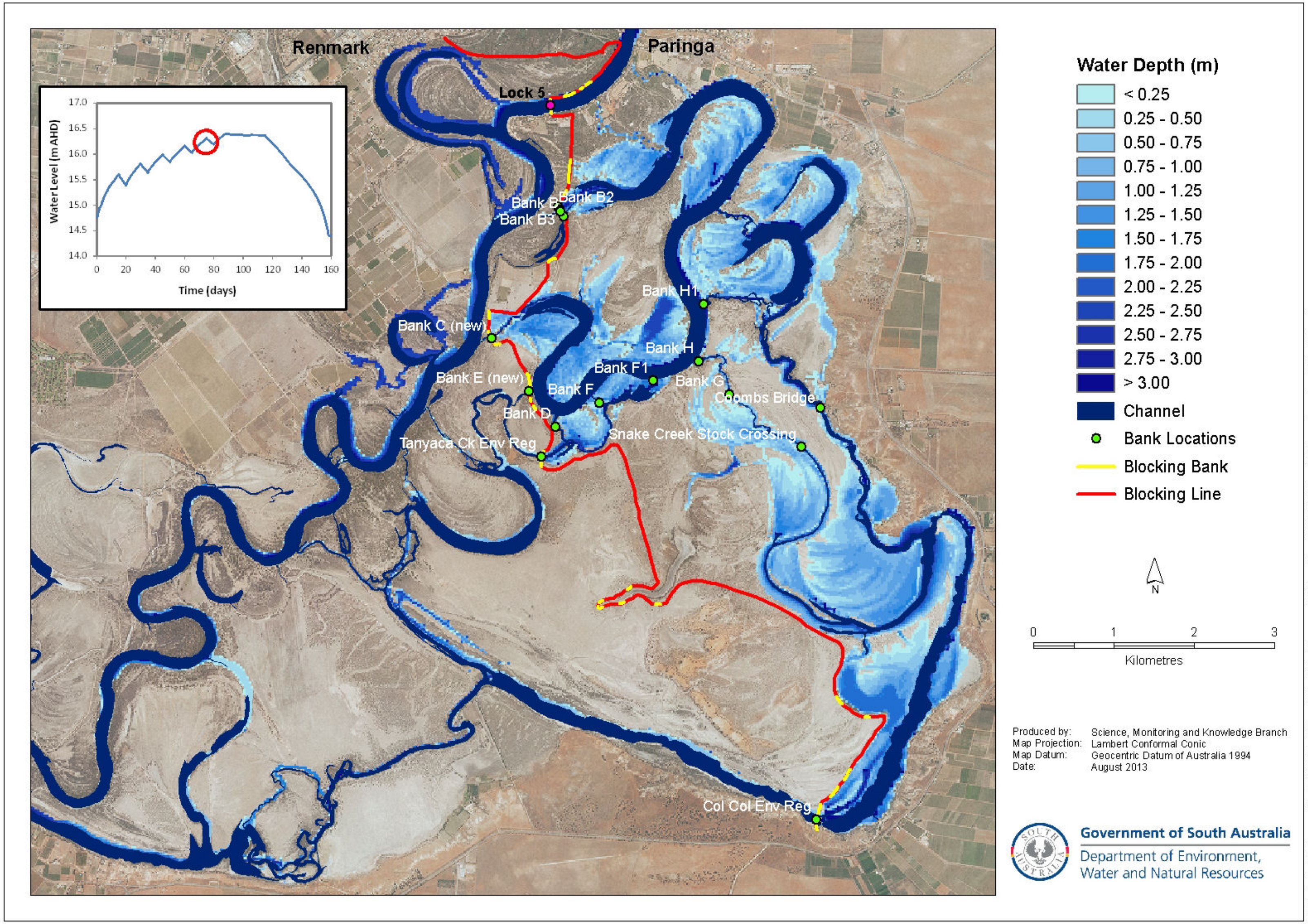

Figure 35: Inundation extent during staggered fill and drain, Day 75 of operation.

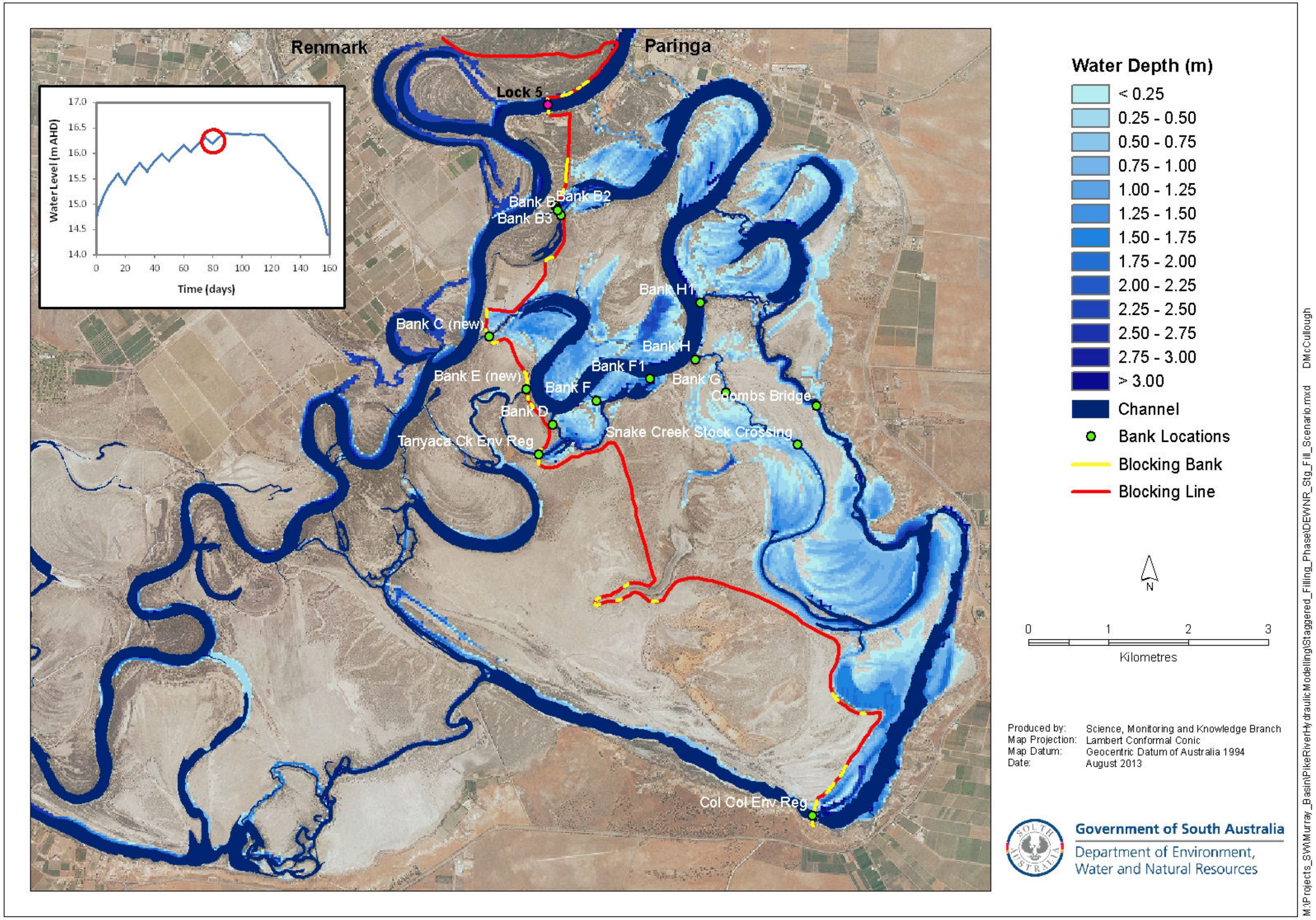

Figure 36: Inundation extent during staggered fill and drain, Day 80 of operation.

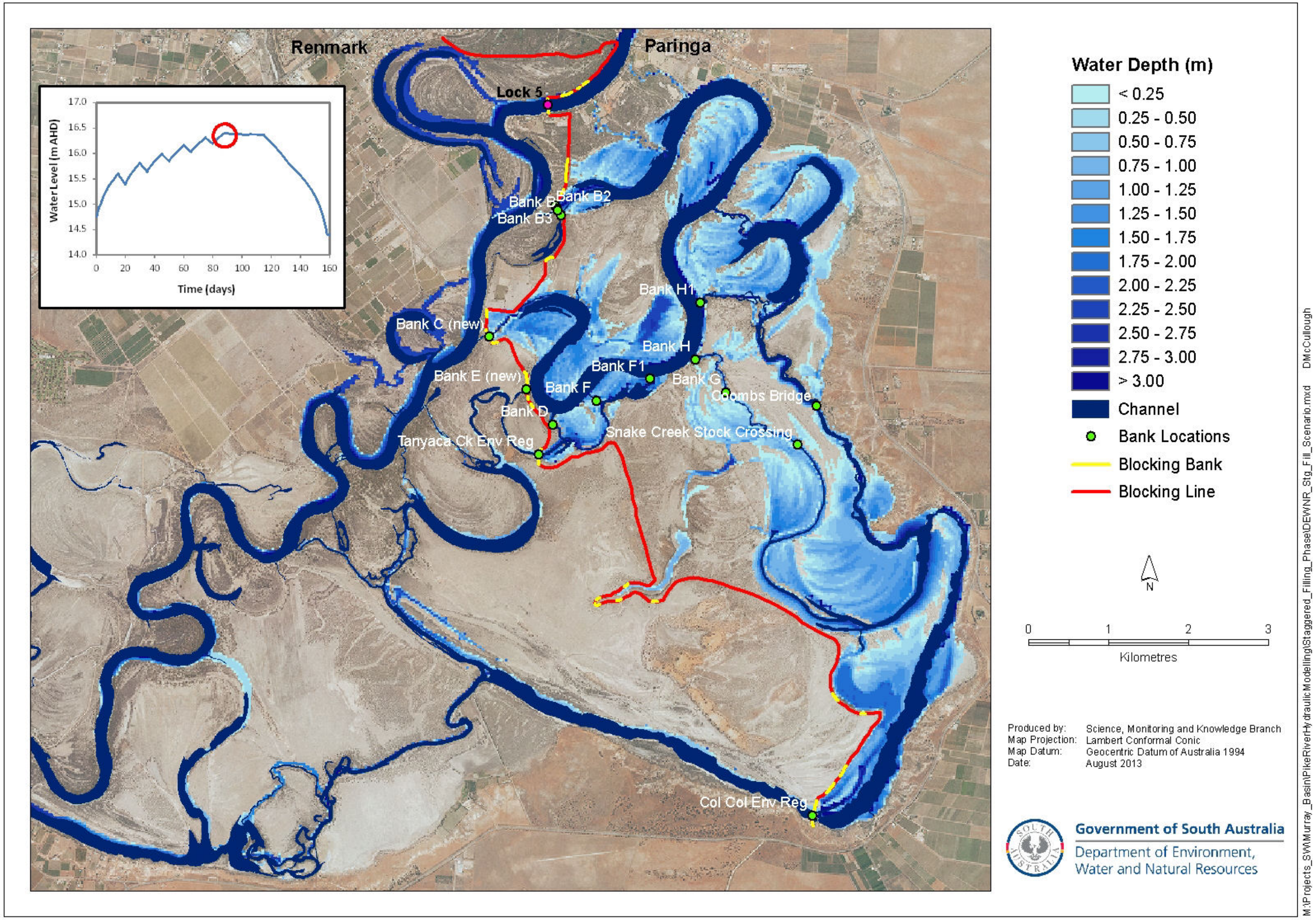

Figure 37: Inundation extent during staggered fill and drain, Day 88 of operation.

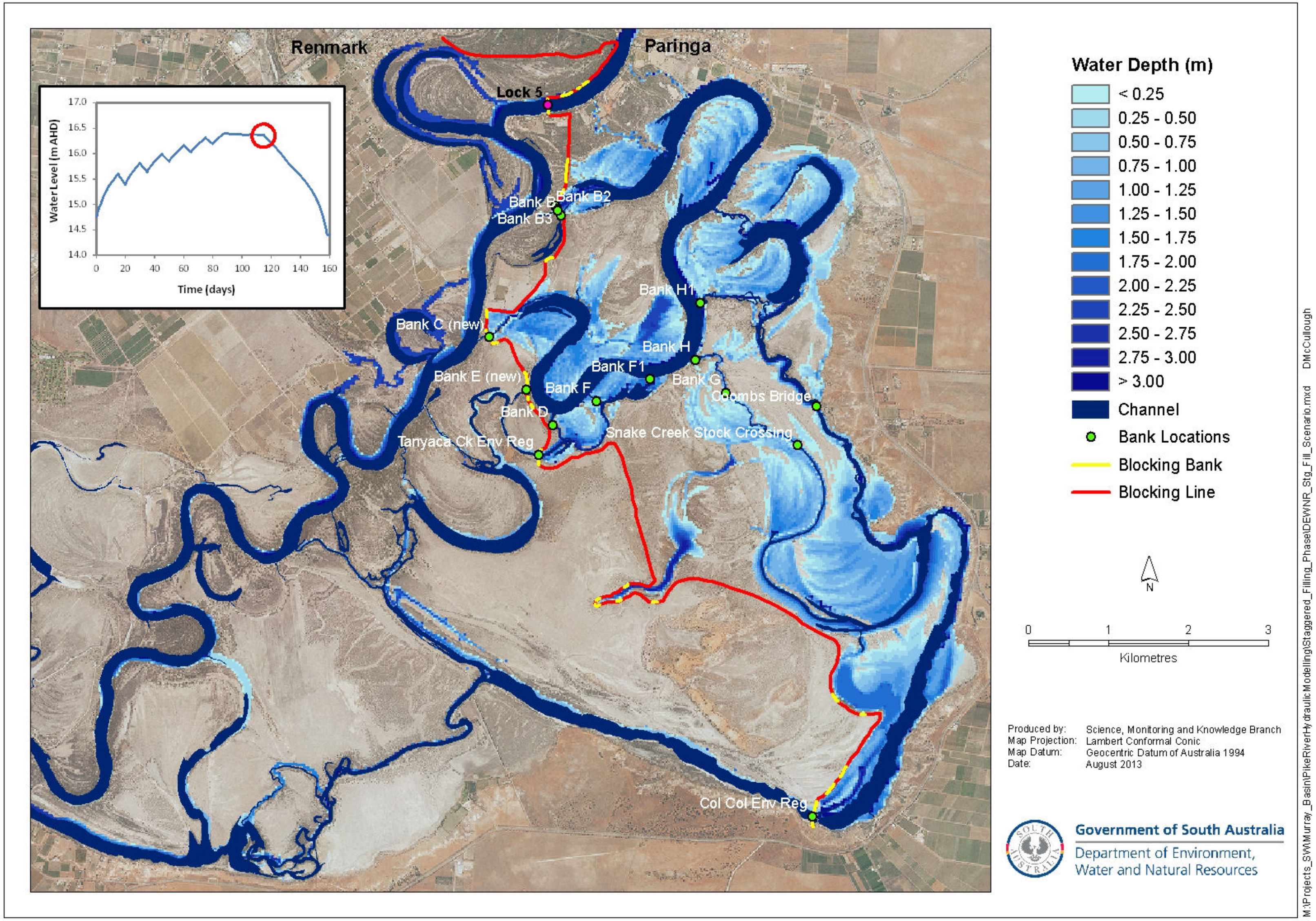

Figure 38: Inundation extent during staggered fill and drain, Day 115 of operation.

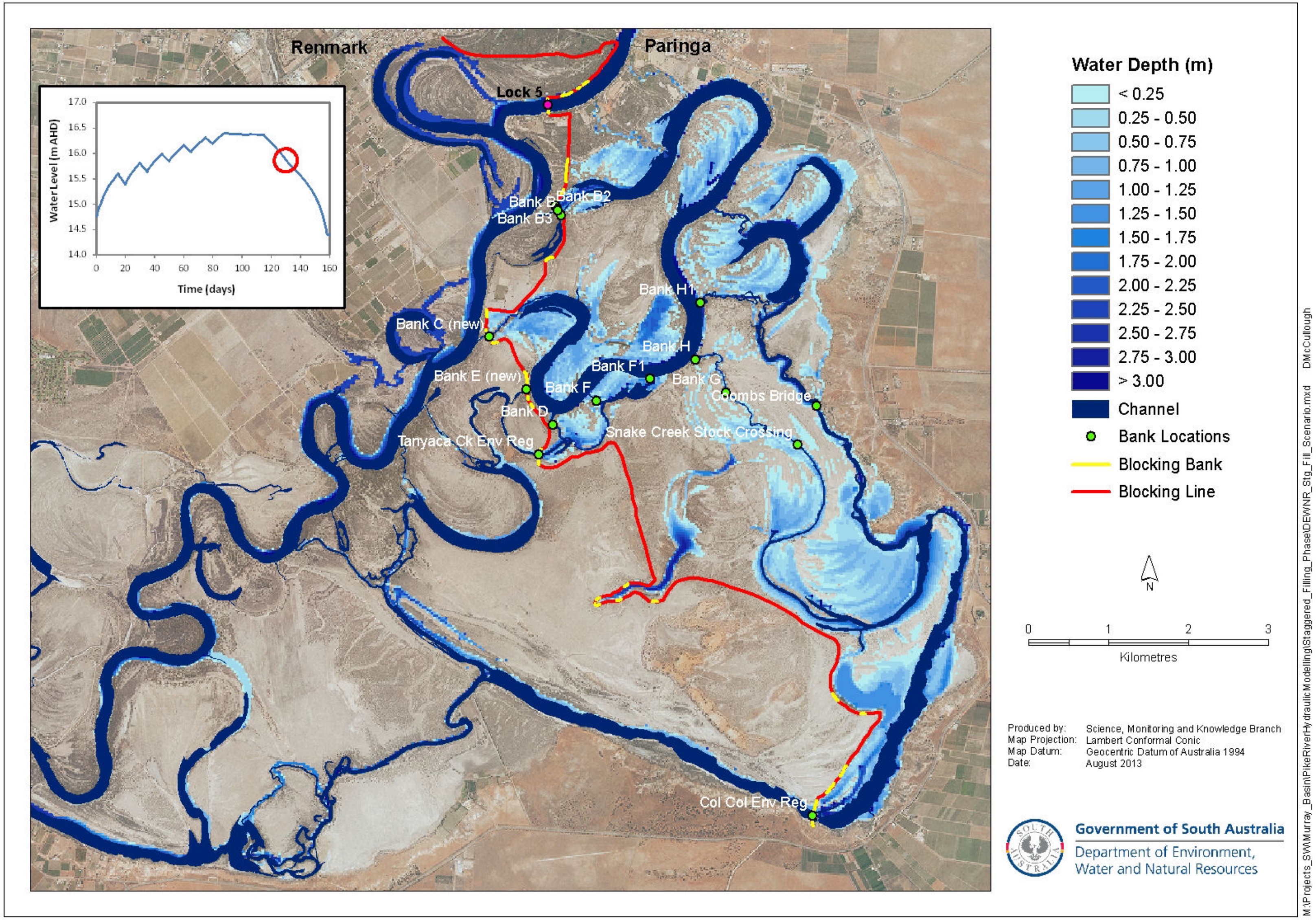

Figure 39: Inundation extent during staggered fill and drain, Day 130 of operation.

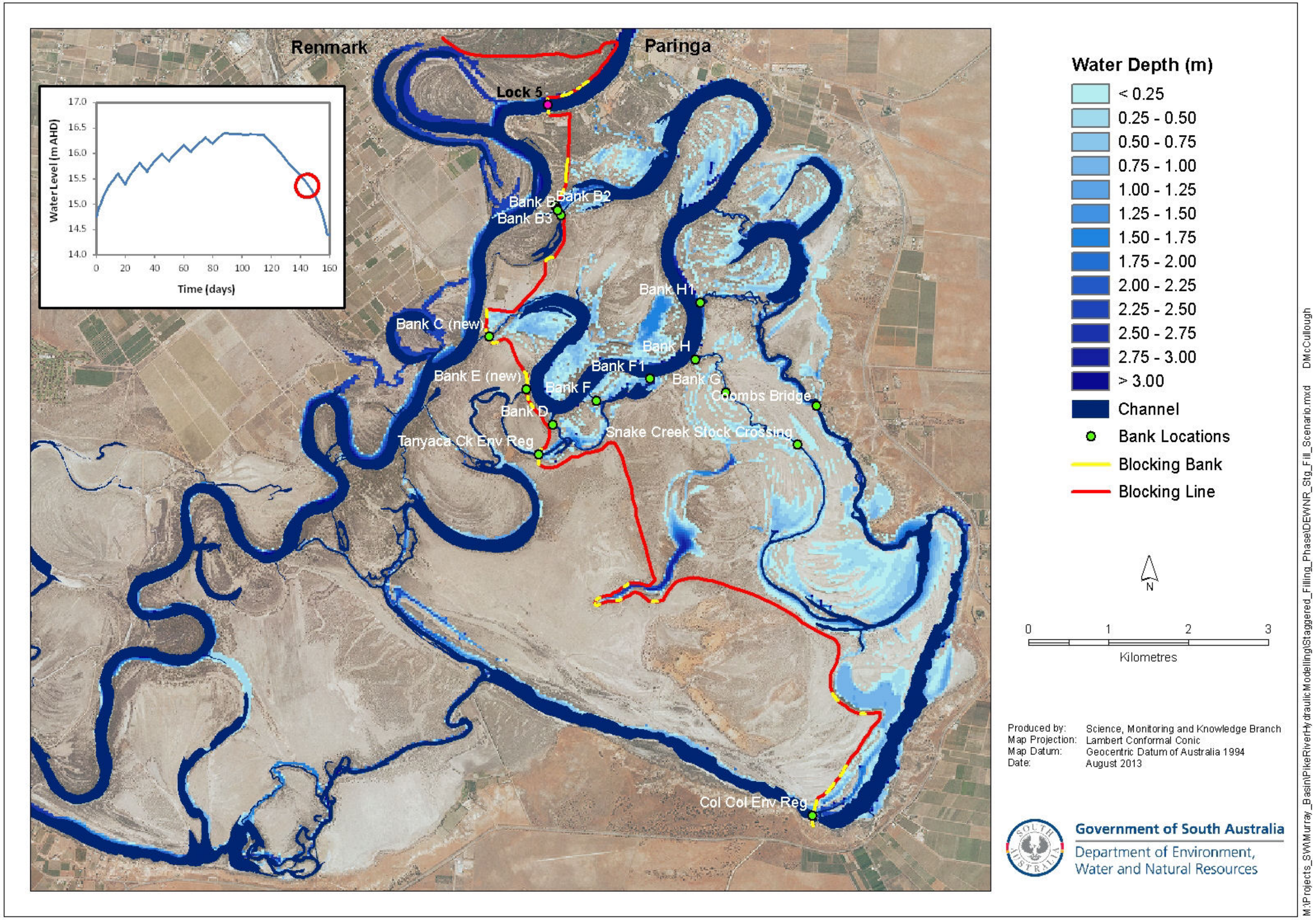

Figure 40: Inundation extent during staggered fill and drain, Day 145 of operation.

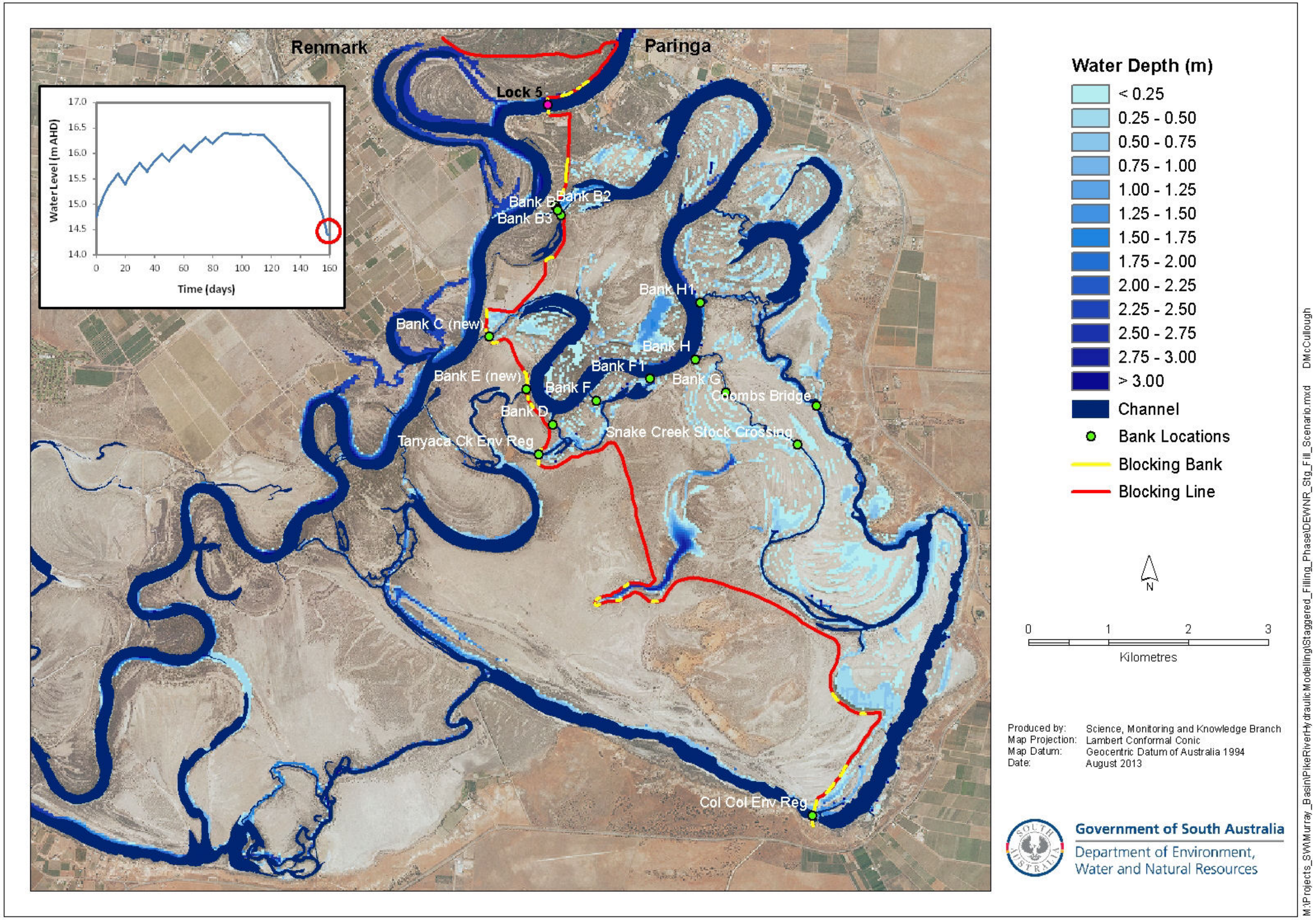

Figure 41: Inundation extent during staggered fill and drain, Day 160 of operation.

Table 8 shows water level, discharge, velocity and bed shear stress at Day 88 (commencement of holding phase, flows over Col Col regulator at 600 ML/d and Tanyaca regulator at 400 ML/d), Day 98 (during holding phase, Col Col at 400 ML/d and Tanyaca at 600 ML/d) and Day 160 (completion of draining phase) for the current scenario at the reporting locations in Figure 1. The results are also compared with results from the previous Water Technology scenario at 30,000 ML/d at Lock 5, noting that only results from Day 44 are reported in those previous results. Velocities highlighted in green indicate velocities ideal for fish passage. Complementary velocity maps are also shown for each of the aforementioned days of operation (Figure 42 to Figure 44), with velocities conducive to fish passage represented by dark orange colouring.

Results are similar to those from Scenario 2, indicating that velocities immediately downstream of both Col Col and Tanyaca regulators are within the fish passage range of 0.18 to 1.4 m/s during both holding and draining phases, especially during the draining phase downstream of the Col Col regulator. Velocities greater than 0.1 m/s are also experienced in Pike River between Mundic Lagoon and upstream of Col Col during the draining phase, promoted by the elevated flow over Col Col to facilitate drainage (i.e. 1400 ML/d).

Bed shear stress remains low at all reported locations relative to the risk of bank erosion, with the highest bed shear stress value being 1.4 N/m^2 in Deep Creek at the end of the draining phase – significantly less than the critical value of 11 N/m^2 as identified previously in this report, beyond which the risk of bank erosion (for clay soils) begins to increase.

5.3 Summary

The operational management regime presented in the preceding chapter indicates that the filling phase of regulator operation can be managed successfully to achieve a 20% exchange of water via a staggered fill-drain operational regime, using only Col Col and Tanyaca environmental regulators as drainage points. For a total inflow of 1,000 ML/d through Deep Creek and Margaret Dowling, each staggered fill requires approximately 575 ML/d combined flow over Col Col and Tanyaca regulators, while each staggered drain requires approximately 1,350 ML/d combined flow.

One of the potential disadvantages with the Scenario 3 operational regime compared to that of Scenario 2 is the intensive and complicated operational requirement over the duration of the managed inundation scheme. During the 90-day filling phase of the hypothetical hydrograph, the Col Col and Tanyaca regulators require manipulation (as a minimum) every ~10 days for each staggered fill period and ~5 days for each staggered draining, while during the holding phase the regulators require manipulation every ~5 days for a further month, before entering the draining phase. Operation is also carried out over a longer period of time (i.e. ~160 days compared to <120 days for Scenario 2), lengthening the operational commitment period by almost 1.5 months. The intended operators of the regulators should be consulted with to ensure that the preceding operational management regime is practical alongside other operational commitments (e.g. Lock 5 operation). Less intensive manipulation may be achieved by lengthening the periods between each staggered fill-drain stage, although this would need to be balanced by the water quality exiting the blocking alignment.

Table 8: Water level, discharge, velocity and bed shear stress at Days 88, 98 and 160 of current operational management scenario compared to existing conditions.

Reporting Location /Stream Name		Previous Scenario, Day 44 30,000 ML/d (full inundation)[1]				Current Scenario, Day 88 (full inundation extent)				Current Scenario, Day 98 (full inundation extent)				Current Scenario, Day 160 (end of draining phase)			
		h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²
1	Deep Creek	16.45	600	0.05	0.27	16.46	600	0.23	0.35	16.43	600	0.23	0.37	15.67	600	0.42	1.36
2	Margaret Dowling	16.41	400	0.08	0.18	16.42	400	0.11	0.08	16.39	400	0.12	0.09	15.06	400	0.30	0.66
3	Mundic Lagoon - Bank B	16.42	0	0.00	0.01	16.41	5	0.00	0.00	16.38	0	0.00	0.00	14.95	1	0.00	0.00
4	Mundic Lagoon	16.42	938	0.02	0.02	16.41	963	0.02	0.00	16.38	998	0.02	0.00	14.94	1004	0.03	0.00
5	Mundic Lagoon Outlet 1	0.00	0	0.10	0.00	16.41	0	0.00	0.00	16.38	0	0.00	0.00	15.96	0	0.00	0.00
6	Mundic Lagoon Outlet 2	16.40	32	0.04	0.00	16.41	165	0.02	0.00	16.38	89	0.01	0.00	14.93	407	0.11	0.08
7	Mundic Lagoon Outlet 3	16.40	27	0.19	0.00	16.41	146	0.02	0.00	16.38	45	0.00	0.00	14.92	532	0.21	0.36
8	Upper Pike River	16.42	224	0.06	0.00	16.41	391	0.01	0.00	16.38	229	0.00	0.00	14.91	491	0.03	0.00
9	Snake Creek - Bank G	16.40	90	0.02	0.00	16.41	172	0.03	0.02	16.38	93	0.02	0.00	14.93	29	0.03	0.02
10	Tanyaca Creek - Bank F1	16.42	145	0.16	0.04	16.41	96	0.00	0.00	16.38	184	0.01	0.00	14.93	43	0.00	0.00
11	Bank D	N/A	N/A	N/A	N/A	16.41	0	0.00	0.00	16.38	89	0.01	0.00	14.93	5	0.00	0.00
12	Mundic Lagoon - Bank C	16.42	3	0.01	0.01	16.41	1	0.00	0.00	16.38	0	0.00	0.00	14.93	0	0.00	0.00
13	Pike River	16.42	350	0.00	0.15	16.40	577	0.01	0.00	16.37	396	0.00	0.00	14.59	1133	0.23	0.19
14	Pike River	16.42	383	0.25	0.01	16.40	535	0.01	0.00	16.37	399	0.01	0.00	14.41	1134	0.10	0.06
15	Lower Pike River*	14.05	3746	0.02	0.33	13.37	487	0.04	0.01	13.39	564	0.04	0.01	13.42	714	0.05	0.02
16	Northern Pike Lagoon	16.42	6	0.20	0.00	16.41	1	0.00	0.00	16.38	4	0.00	0.00	14.91	0	0.00	0.00
17	Rumpagunyah*	N/A	N/A	N/A	N/A	13.37	88	0.03	0.01	13.39	437	0.13	0.12	13.39	471	0.14	0.14
18	Pike River Outlet*	N/A	N/A	N/A	N/A	13.31	461	0.02	0.01	13.32	479	0.03	0.01	13.32	597	0.03	0.01
19	Inlet U/S EC pontoon*	N/A	N/A	N/A	N/A	13.30	76	0.02	0.01	13.31	84	0.02	0.01	13.31	118	0.03	0.01
20	Swift Creek*	N/A	N/A	N/A	N/A	13.43	0	0.00	0.00	13.44	0	0.00	0.00	13.44	0	0.00	0.00
21	Wood Duck Creek*	N/A	N/A	N/A	N/A	13.42	0	0.00	0.00	13.43	0	0.00	0.00	13.44	0	0.00	0.00
22	Tanyaca Ck u/s of Tanyaca Lagoon*	14.14	601	0.19	0.32	13.39	50	0.08	0.08	13.67	600	0.37	1.34	13.41	50	0.07	0.06

[1]Reported in previous Water Technology modelling results (locations 11 and 17 to 21 were not reported in those results). *Locations downstream of blocking bank.

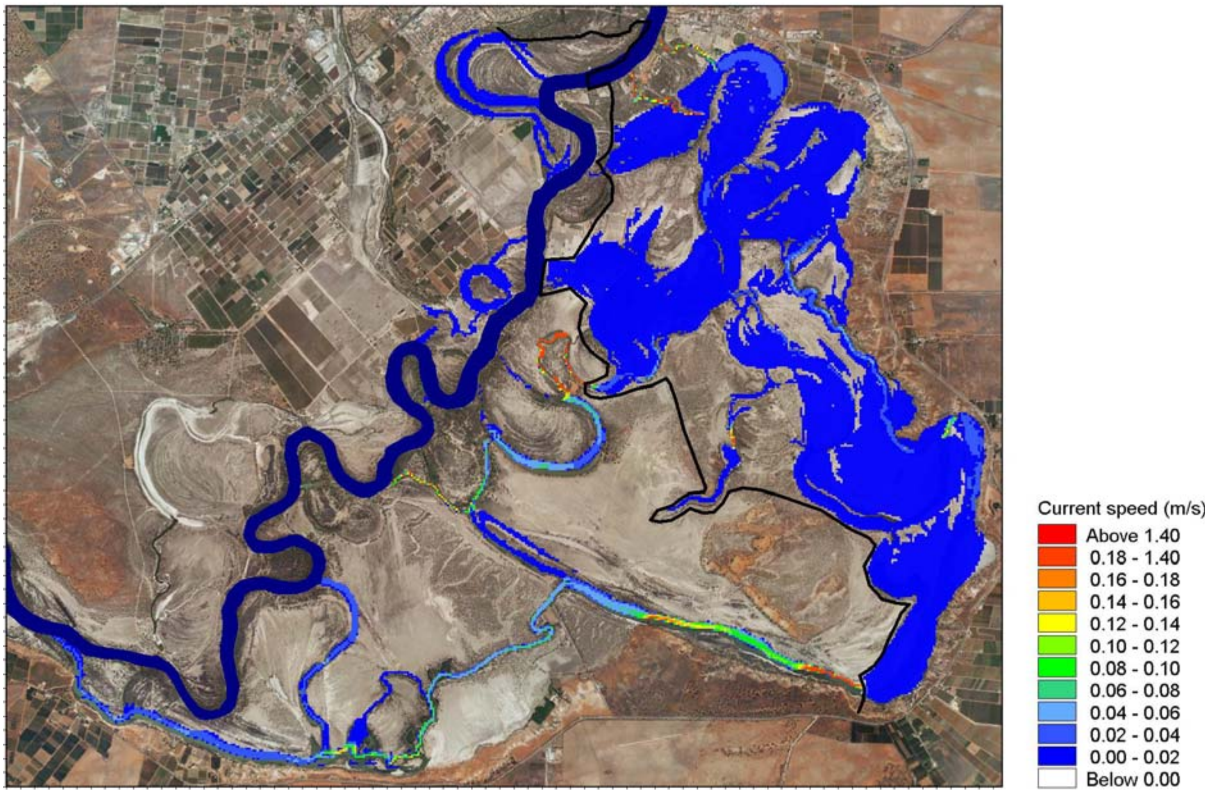

Figure 42: Velocity map at Day 88 of operational management scenario (start of holding phase).

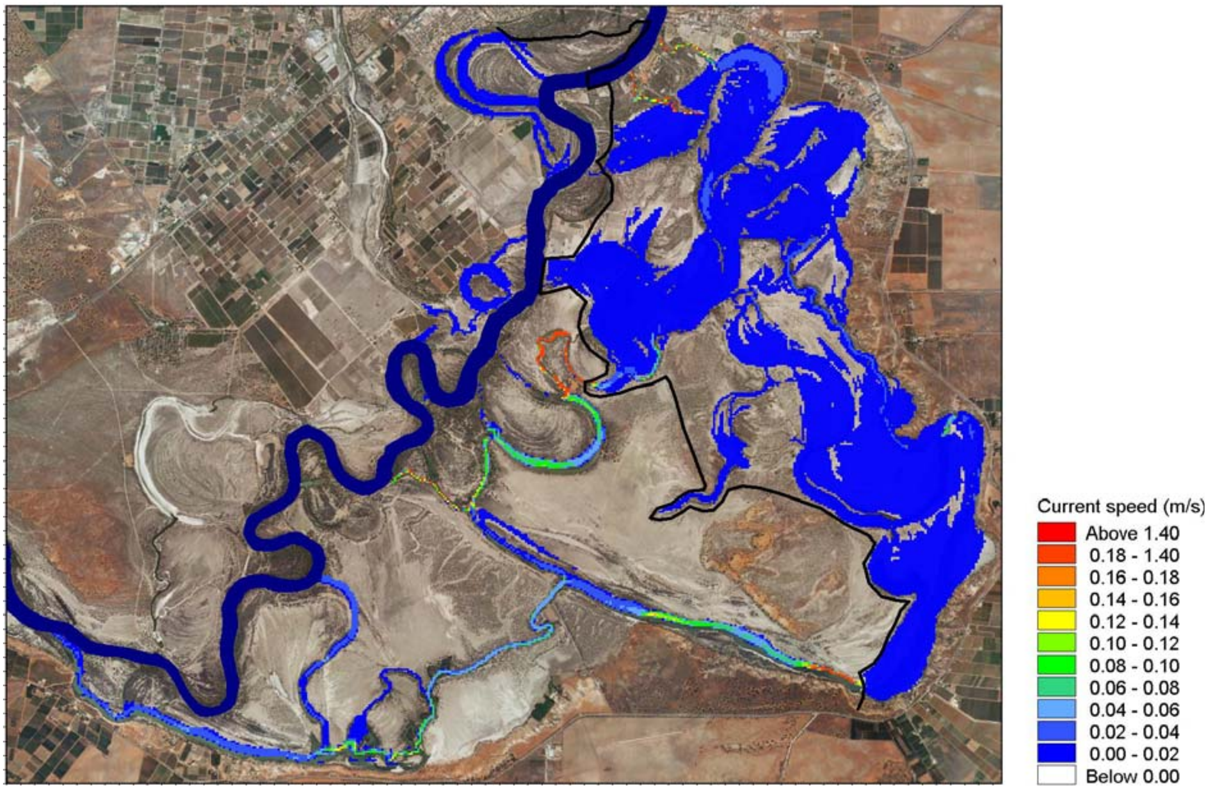

Figure 43: Velocity map at Day 98 of operational management scenario (during holding phase).

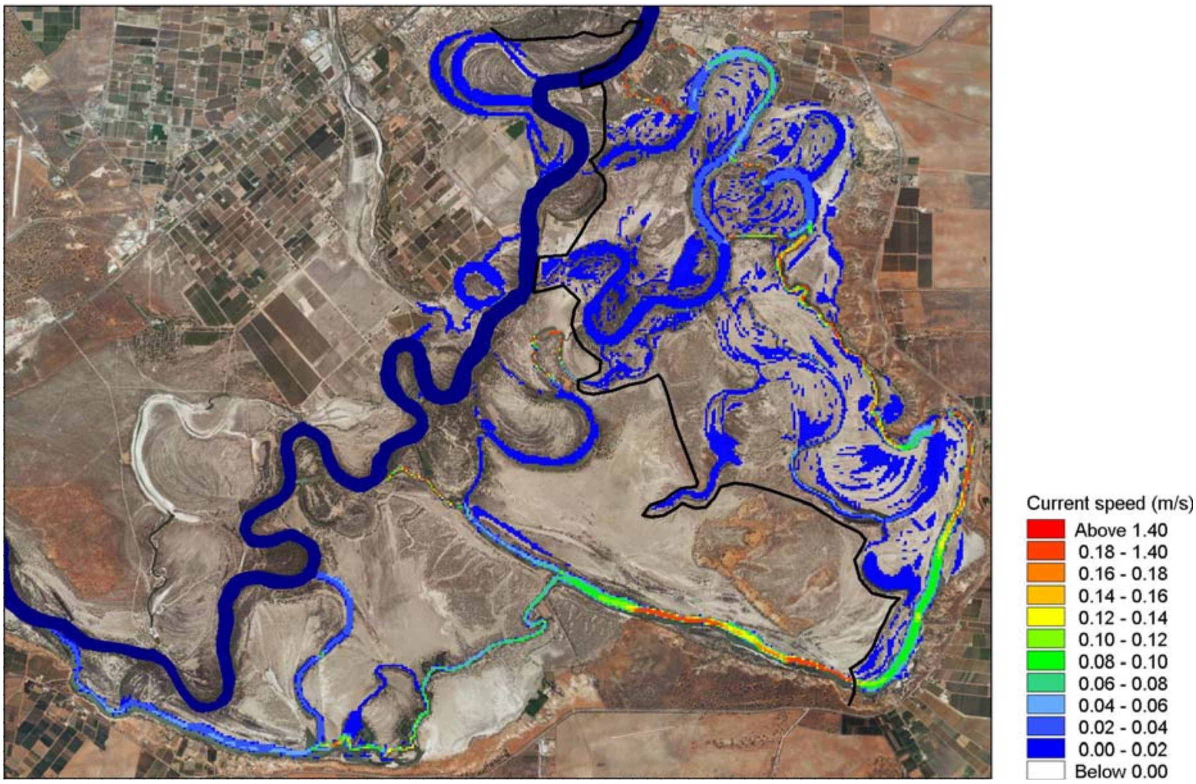

Figure 44: Velocity map at Day 160 of operational management scenario (end of draining phase).

6. Scenario 4 – Mundic Lagoon Operating Level with Controlled Flow over Bank D

The following sections present results from a modelling simulation to examine the effect of design flows through new control structures at Banks D and F1 on water level in Mundic Lagoon at 10,000 ML/d flow at Lock 5. The results are to be used to inform engineering design calculations for the new structures.

6.1 Model Simulation

The simulation was executed using the following system configuration:

- Flow over Lock 5 at 10,000 ML/d
- Lock 5 U/S level at 16.3 mAHD (i.e. normal pool level)
- Total inflow to the system at 1,000 ML/d (600 ML/d at Deep Creek and 400 ML/d at Margaret Dowling)
- Banks B and C closed
- Flow over Bank D = 500 ML/d
- Flow over Bank F1 = 50 ML/d.

Reporting locations are shown in Figure 1, with results presented in Table 9 compared to modelled results at existing conditions (note that flows through Margaret Dowling and Deep Creek are reduced in the existing conditions compared to the current scenario). Figure 45 additionally presents a velocity map of the anabranch under steady state conditions of the model output.

6.2 Results and Discussion

When operating at 500 ML/d over Bank D and 50 ML/d over Bank F1 the results indicate that the level in Mundic Lagoon upstream of Bank D is maintained at approximately 14.7 m AHD, with water level downstream of Bank D at approximately 13.6 m AHD (Table 9). The upstream level is only marginally lower than the normal operating level in Mundic Lagoon (i.e. 14.75 m AHD), indicating that operating at such flows will not adversely affect the hydraulic condition of Mundic Lagoon.

Velocities conducive to fish passage (i.e. 0.18 to 1.4 m/s) are shown to persist through Tanyaca Creek into Mundic Lagoon via Bank D at these flow conditions (Figure 45).

Table 9: Water level, discharge, velocity and bed shear stress for controlled flow over Bank D (500 ML/d) and Bank F1 (50 ML/d) at Lock 5 flow of 10,000 ML/d and U/S water level at 16.3 m AHD (see Figure 1 for reporting locations).

Reporting Location /Stream Name		Existing Conditions – 10,000 ML/d				Current Scenario – 10,000 ML/d			
		h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²
1	Deep Creek	15.31	182	0.19	0.95	16.01	600	0.32	2.27
2	Margaret Dowling	14.82	170	0.16	0.90	15.90	400	0.43	6.98
3	Mundic Lagoon - Bank B	14.52	0	0.00	0.00	14.70	0	0.00	0.00
4	Mundic Lagoon	14.52	323	0.02	0.00	14.70	964	0.05	0.02
5	Mundic Lagoon Outlet 1	14.52	16	0.03	0.01	14.70	34	0.04	0.02
6	Mundic Lagoon Outlet 2	14.52	154	0.06	0.02	14.69	204	0.06	0.03
7	Mundic Lagoon Outlet 3	14.52	136	0.08	0.05	14.69	176	0.09	0.06
8	Southern Pike Lagoon	14.52	154	0.01	0.00	14.69	226	0.01	0.00
9	Snake Creek - Bank G	14.52	0	0.00	0.00	14.70	0	0.00	0.00
10	Tanyaca Creek - Bank F1	14.52	0	0.00	0.00	14.69	50	0.14	0.26
11	Bank D	14.52	0	0.00	0.00	14.68	500	0.20	0.36
12	Mundic Lagoon - Bank C	14.52	4	0.00	0.00	14.70	0	0.00	0.00
13	Pike River	14.40	271	0.04	0.01	14.49	390	0.06	0.02
14	Pike River	14.40	254	0.03	0.01	14.49	378	0.04	0.01
15	Lower Pike River	13.29	1182	0.09	0.05	13.31	1472	0.11	0.07
16	Northern Pike Lagoon	14.52	15	0.00	0.00	14.69	34	0.00	0.00
17	Rumpagunyah	13.33	674	0.21	0.31	13.35	345	0.11	0.08
18	Pike River Outlet	13.26	1385	0.06	0.02	13.26	1664	0.07	0.02
19	Inlet U/S EC pontoon	13.28	212	0.06	0.05	13.29	192	0.06	0.04
20	Swift Creek	13.39	199	0.23	0.48	13.40	144	0.16	0.24
21	Wood Duck Creek	13.41	68	0.13	0.19	13.41	55	0.11	0.13
22	Tanyaca Ck u/s of Tanyaca Lagoon	13.32	0	0.00	0.00	13.62	550	0.37	1.41

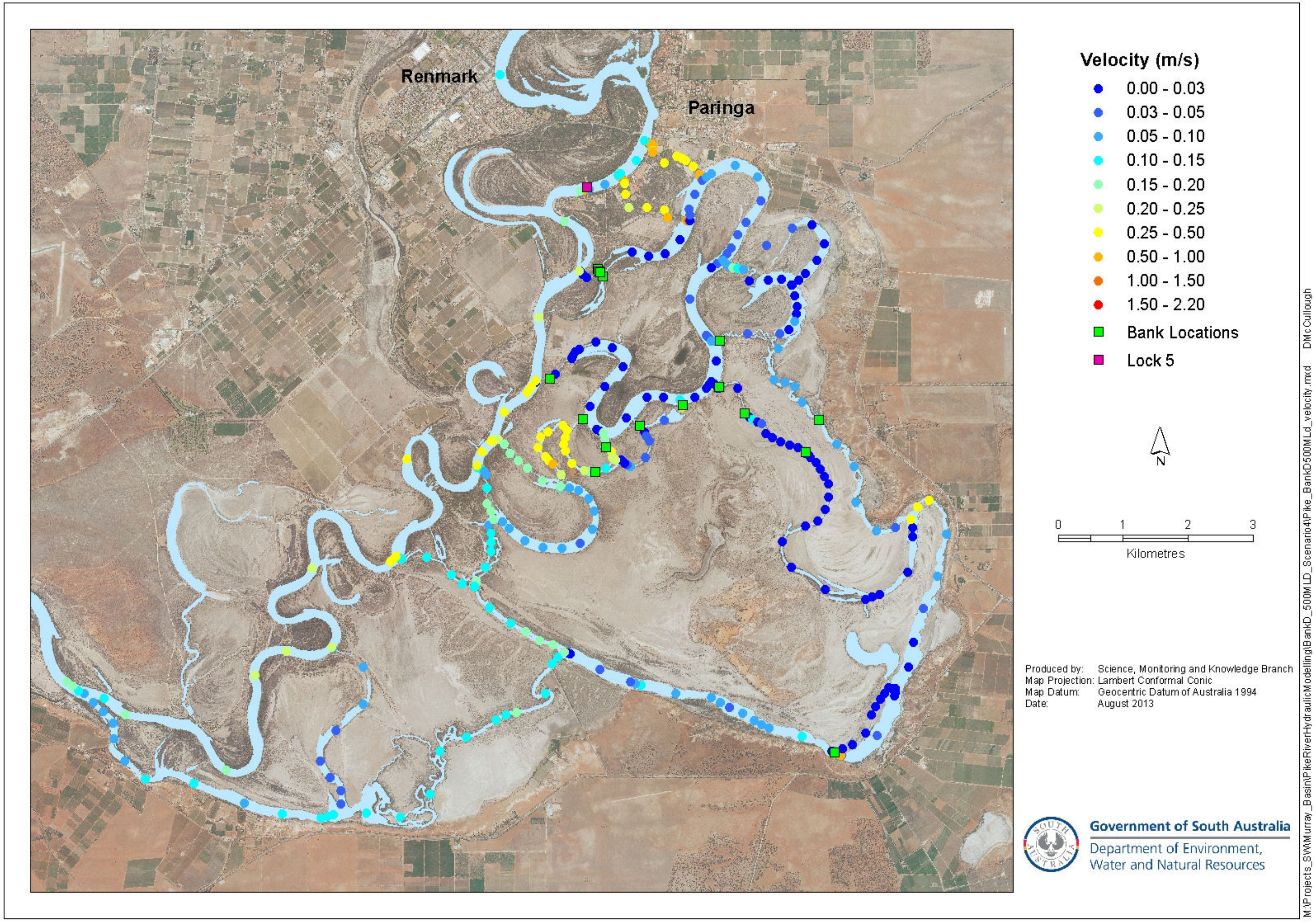

Figure 45: Velocity map for controlled flow over Bank D (500 ML/d) and Bank F1 (50 ML/d) at Lock 5 flow of 10,000 ML/d and U/S water level at 16.3 mAHD.

7. Scenario 5 – Flow Capacity at Bank D with Upgraded Infrastructure at 50,000 ML/d Lock 5 Flow

The following scenario examines the potential flow capacity at Bank D under high flow conditions in the River Murray (approximately 50,000 ML/d at Lock 5), with 1000 ML/d flow through Margaret Dowling and Deep Creek, and Banks B and C open to allow additional inflow from the River Murray. Results are to be used to inform engineering designs such that the Bank D structure can be configured to pass the maximum theoretical flow.

7.1 Model Simulation

The model was executed using the following parameters:

- 50,000 ML/d flow at Lock 5;
- Water levels upstream of Lock 4 at 13.85 mAHD and downstream of Lock 5 at 15.93 m AHD (based on lock levels monitored during Lock 5 gauging at approximately 50,000 ML/d);
- 1,000 ML/d combined flow entering through Deep Creek (600 ML/d) and Margaret Dowling (400 ML/d);
- Flow over Banks B and C controlled as close as practicable to 1,500 ML/d to maintain appropriate velocity profiles (N.B. simulated flows of 1,520 ML/d and 1,632 ML/d were achieved at Banks B and C, respectively);
- Bank D removed, with no further flow manipulation via structures connecting Mundic Lagoon to Tanyaca Creek and Pike River;
- All banks upgraded as per latest detailed designs (as at February 2013); and
- Environmental regulators at Tanyaca and Col Col are <u>not</u> constructed.

Note that modifications were made to the model, compared to that used for previous scenarios presented, to include updated bank survey data where applicable.

Results are presented at steady state conditions, including:

- Table of water level, flow, velocity and bed shear stress at the standard reporting locations, including flows at all relevant flow control structures (N.B. velocities in the table highlighted in green indicate ideal fish habitat conditions).
- Map indicating the inundation extent over the floodplain at steady flow.
- Map indicating velocities through the anabranch.

Reporting locations for tabulated results are shown in Figure 1.

7.2 Results and Discussion

The simulation results, shown in Table 10, indicate that the upgraded structures, coupled with high rates of flow in the River Murray (50,000 ML/d), has resulted in a general increase in water level throughout the anabranch relative to existing conditions. This has resulted in flows under upgraded conditions at the reporting locations being generally equivalent to or higher than those under existing conditions.

Flow of approximately 2,185 ML/d is shown to pass over Bank D at full capacity, which is approximately 200% greater than that shown under existing conditions (i.e. bank configurations pre-upgrade) at 50,000 ML/d Lock 5 flow. This high flow over Bank D corresponds to a velocity of 0.36 m/s and bed shear stress of 0.75 N/m^2. The relatively low shear stress value indicates that no significant bank erosion is expected to occur under the simulated flow conditions. Note however that reported velocities and shear stresses represent average values over each relevant cross-section, and as such localised values will potentially exceed these values and may promote local scour directly below the Bank D structure.

Flows out of Mundic Lagoon through other connected locations in the Pike anabranch include approximately 340 ML/d at Bank F1, 160 ML/d through Snake Creek, and 670 and 680 ML/d through Mundic Lagoon Outlets 2 and 3, respectively. These flows are all increased relative to existing conditions with the exception of Bank F1, which is reported as approximately 490 ML/d under pre-upgrade conditions at 50,000 ML/d. This difference is a result of updating the model configuration at the Bank F1 location, which has resulted in a rise in the reference level at the top of the bank.

Inflows to the anabranch at the Bank B complex is shown as essentially no flow under existing conditions and over 1,500 ML/d under the upgraded model configuration. Again, this difference can be attributed to the revision of the model to include updated survey results and the latest design of Bank B, which has allowed a much greater capacity through this section of the anabranch complex. One issue identified with the model in its present configuration is that the connection between the River Murray and Bank B is represented by the 2D section of the model (i.e. 30 m grid size), which is not ideal for adequately representing this stream given its width is less than 30 m. This may result in a greater passage of flow into Bank B from the River Murray than would be realistically possible. It is recommended that this stream be replaced by a 1D representation in future studies to more accurately define this section of the anabranch.

The velocity map (Figure 47) indicates ideal fish passage conditions (i.e. 0.18 to 1.4 m/s) through the lower sections of the Pike River, extending up into Tanyaca Creek, through Bank D and into Mundic Lagoon. Acceptable velocities are also observed through Swift and Wood Duck Creeks, Banks B and C, and in sections of the Pike River upstream of Col Col bank. Some differences in velocity distribution at the reporting locations are observed when compared to existing conditions (Table 10) however a full assessment of changes in velocity distribution is not possible without remodelling existing conditions, given that a number of locations were not reported on in the previous Water Technology results.

7.3 Summary

Approximately 2,185 ML/d passes through upgraded Bank D at full capacity under high flow conditions (i.e. 50,000 ML/d at Lock 5, over 1,500 ML/d entering the anabranch through Banks B and C, and 1,000 ML/d at Margaret Dowling and Deep Creek). Reported bed shear stress indicates no significant erosion will occur under such flow, although it is likely that some bank protection will be required downstream of the upgraded Bank D regulator in particular to protect from localised scouring. The results indicate that ideal fish passage conditions are promoted under the simulated flow conditions, in particular through Bank D into Mundic Lagoon.

Table 10: Water level, discharge, velocity and bed shear stress for 50,000 ML/d flow at Lock 5, Banks B, C and D fully open (see Figure 1 for reporting locations).

Reporting Location /Stream Name		Pre-upgraded Conditions – 50,000 ML/d				Current Scenario – 50,000 ML/d			
		h m AHD	Q ML/d	v m/s	τ N/m²	h m AHD	Q ML/d	v m/s	τ N/m²
1	Deep Creek	15.71	600	0.00	0.95	15.79	600	0.38	1.07
2	Margaret Dowling	15.28	400	0.07	1.31	15.52	400	0.20	0.29
3	Mundic Lagoon - Bank B	15.23	1	0.00	0.00	15.51	1,520	0.05	0.02
4	Mundic Lagoon	15.23	993	0.03	0.03	15.49	2,520	0.08	0.03
5	Mundic Lagoon Outlet 1	0.00	0.00	0.03	0.00	Dry	-	-	-
6	Mundic Lagoon Outlet 2	15.22	582	0.04	0.38	15.47	674	0.13	0.10
7	Mundic Lagoon Outlet 3	15.24	493	0.25	0.17	15.46	680	0.17	0.21
8	Southern Pike Lagoon	15.21	581	0.05	0.00	15.45	674	0.02	0.00
9	Snake Creek - Bank G	15.23	0.00	0.01	0.00	15.46	164	0.07	0.12
10	Tanyaca Creek - Bank F1	15.21	489	0.39	0.01	15.44	341	0.02	0.00
11	Bank D	N/A*	741*	N/A*	N/A*	15.45	2,185	0.36	0.75
12	Mundic Lagoon - Bank C	15.28	1390	0.23	0.01	15.48	1,632	0.35	0.86
13	Pike River	15.06	1174	0.15	0.31	15.26	1,516	0.22	0.23
14	Pike River	15.04	853	0.49	0.01	15.21	1,516	0.06	0.02
15	Lower Pike River	14.93	9905	0.00	1.25	15.07	10,478	0.45	1.00
16	Northern Pike Lagoon	15.21	0.00	0.14	0.00	15.45	0	0.00	0.00
17	Rumpagunyah	N/A*	1028*	N/A*	N/A*	15.22	105	0.01	0.00
18	Pike River Outlet	N/A*	N/A*	N/A*	N/A*	14.79	9,085	0.25	0.44
19	Inlet U/S EC pontoon	N/A*	N/A*	N/A*	N/A*	14.90	1,182	0.32	0.70
20	Swift Creek	N/A*	4052*	N/A*	N/A*	15.38	3,696	0.35	0.77
21	Wood Duck Creek	N/A*	2436*	N/A*	N/A*	15.45	2,512	0.50	1.52
22	Tanyaca Ck u/s of Tanyaca Lagoon	15.16	1230	0.36	0.14	15.35	2,633	0.26	0.49

*: Values from Water Technology. Limited values reported for locations 11 and 17 to 21, full data for these locations were not included in the original Water Technology reporting.

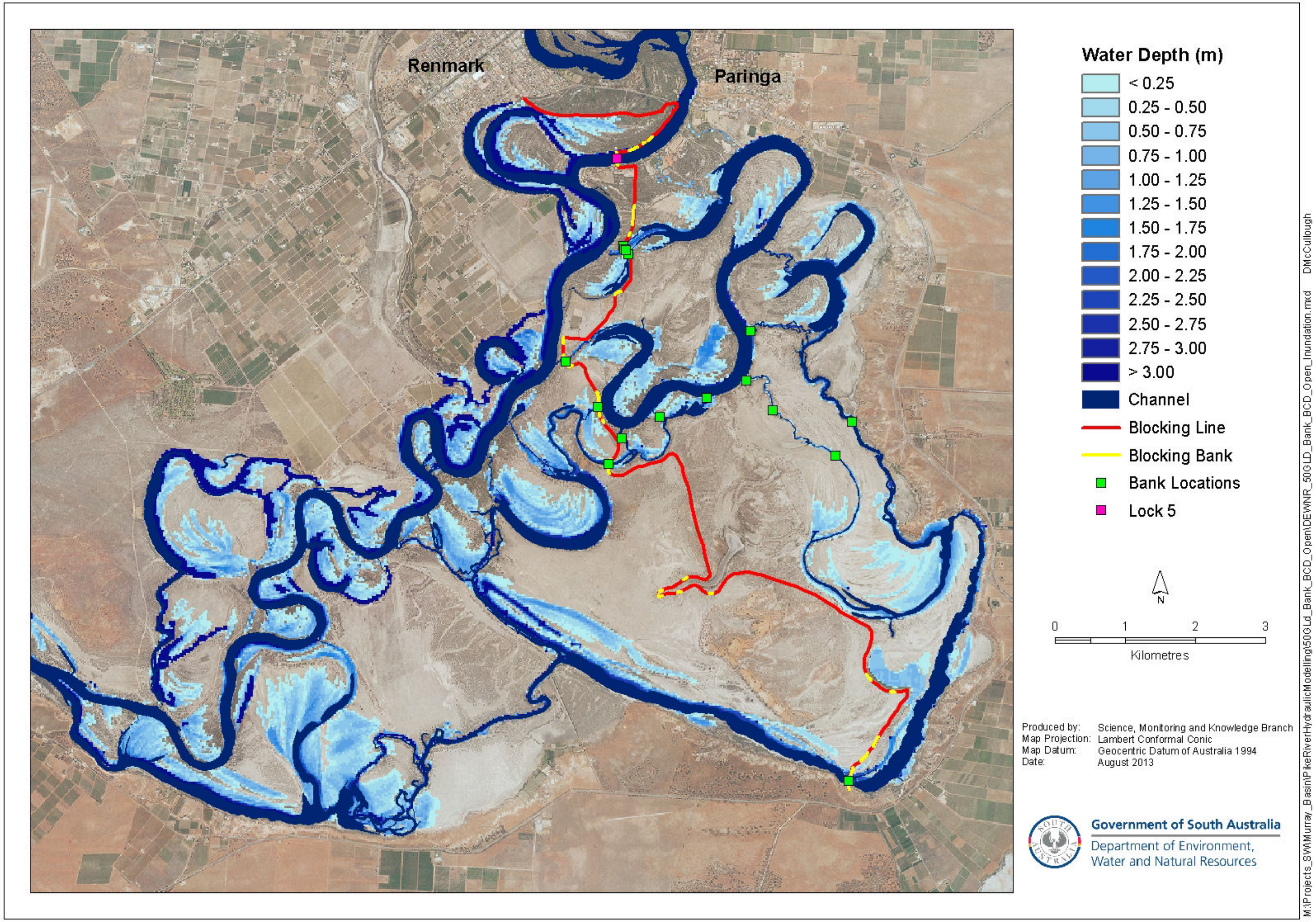

Figure 46: Inundation extent at 50,000 ML/d flow at Lock 5, Banks B and C open.

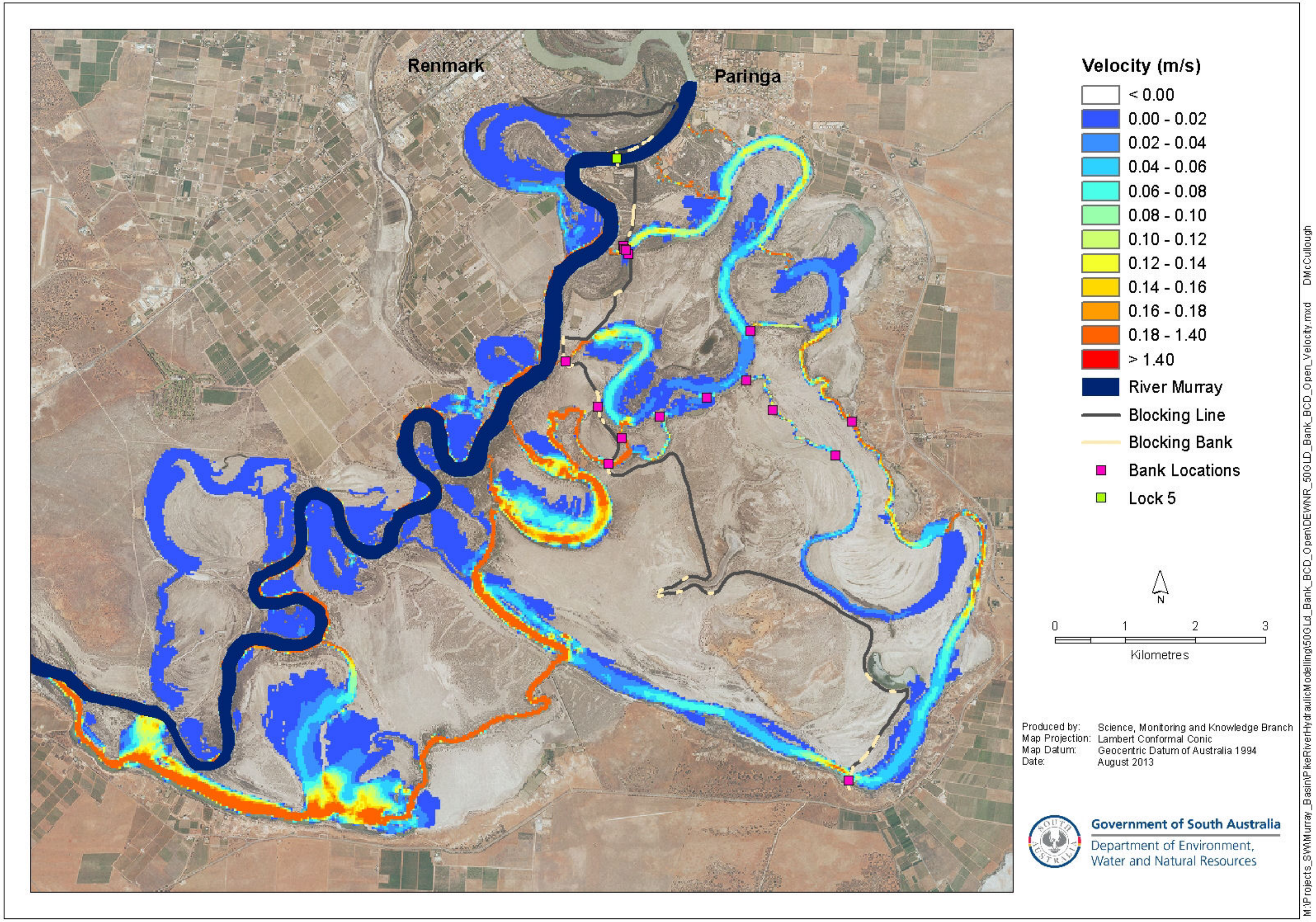

Figure 47: Velocity map at 50,000 ML/d flow at Lock 5, Banks B and C open.

8. Scenario 6 – Inundation Extent and Velocity Changes Downstream of Banks D, F and F1 under Alternative Control Schemes

The following scenario compares changes in inundation extent and water velocity in the Pike floodplain complex under various control schemes comprising Banks D, F, F1 and the proposed Tanyaca Creek Environmental Regulator. The simulation is designed to examine the change in hydrological and ecological condition of the area downstream of Banks D, F and F1 if these controls were removed and the proposed Tanyaca Creek environmental regulator was alternatively used as a single control point. The results are intended to be used to inform further ecological analysis, which will enable a preferred control option to be selected.

8.1 Model Simulation

The MIKE FLOOD model used in previous scenarios was reschematised to a 10 x 10 m grid in order to increase the resolution of the inundation map and improve area calculations. The main actions performed in the development of the new model scheme are as follows:

- Developed a topographic map from a 2008 DEM (Digital Elevation Model), limiting its boundaries to the area encompassing Banks D, F, F1, and the proposed Tanyaca Regulator site only (refer to Figure 48).
- The DEM used for the topographic map contains missing data in areas typically inundated by water, requiring estimation of the bathymetry using downstream invert levels obtained from recent surveys of each bank location. Minimum elevation level downstream of Bank F1 was estimated at 13.6 m AHD, and downstream of Bank D at 12.2 m AHD (N.B. depths calculated in these estimated areas should be considered as indicative only, and may not represent actual depths for the hydrological conditions assessed).
- Coupled the existing MIKE 11 (1-D) model to the topographical grid in MIKE FLOOD (Figure 48), which involved:
    - Removing branches downstream of Banks D, F and F1, and upstream of the proposed Tanyaca Regulator site to allow connectivity between the controls to be represented by the 2-D grid.
    - Linking the free ends of the 1-D model at Banks D, F, F1, and Tanyaca Regulator to the corresponding 2-D grid points.
    - Creating a boundary at Tanyaca Regulator to represent the blocking bank alignment (arbitrary elevation of 100 m AHD) and ensure that all flow upstream of the regulator transitions back into the 1-D section of the model.
- Channel cross-sections in the existing 1-D model were updated with the most recent survey data.
- Hydrological parameters used for the previous hydraulic model were preserved, including roughness and flooding/drying depths.

Four scenarios were tested using the aforementioned model as follows:

Scenario 6A – Current Conditions ('Existing')

- 10,000 ML/d at Lock 5
- Banks D, F and F1 set to current (pre-upgrade) configurations
- Flows into Margaret Dowling and Deep Creek flowing at typical flows under the current (pre-upgrade) configuration (i.e. approximately 170 ML/d each creek)
- No control definitions used on any of the banks; system allowed to reach steady state.

Scenario 6B – Upgraded Banks D, F and F1 ('Upgraded'), Flow through Bank D only

- 10,000 ML/d at Lock 5
- Control definitions setting 1,000 ML/d combined total flow through Margaret Dowling (400 ML/d) and Deep Creek (600 ML/d)
- Banks D, F and F1 upgraded as per current design drawings
- Control definitions set to 500 ML/d at Bank D and Bank F1 closed.

Scenario 6C – Upgraded Banks D, F and F1 ('Upgraded'), Flow through Banks D and F1

- 10,000 ML/d at Lock 5
- Control definitions setting 1,000 ML/d combined total flow through Margaret Dowling (400 ML/d) and Deep Creek (600 ML/d)
- Banks D, F and F1 upgraded as per current design drawings
- Control definitions set to 450 ML/d at Bank D and 50 ML/d at Bank F1.

Scenario 6D – Tanyaca Environmental Regulator as Control ('Alternative')

- 10,000 ML/d at Lock 5
- Control definitions setting 1,000 ML/d combined total flow through Margaret Dowling (400 ML/d) and Deep Creek (600 ML/d)
- Banks D, F and F1 removed, with flow through each location allowed to reach equilibrium (i.e. no control definitions at these sites)
- Control definition at Tanyaca regulator site set to 500 ML/d.

Maps of inundation extent and water velocity are presented and relative areas of inundation under 'existing' (i.e. current bank configurations), 'upgraded' (i.e. Banks D and F1 as controls), and 'alternative' (i.e. Tanyaca Creek Environmental Regulator as the main control) operating methodologies are calculated and analysed.

Inundation extent was calculated using output from the 2-D section of the model, covering the area between Banks D, F, F1, and the proposed Tanyaca Regulator site. Depths were differentiated by colour in 0.1 m increments up to 0.5 m, as requested for the subsequent ecological analysis. Velocity maps include the 2-D section of the model and also spot velocities calculated in the 1-D model sections, allowing measurement of approximate reach lengths (in the creeks between locations i to iv in Figure 48) containing acceptable fish passage velocities.

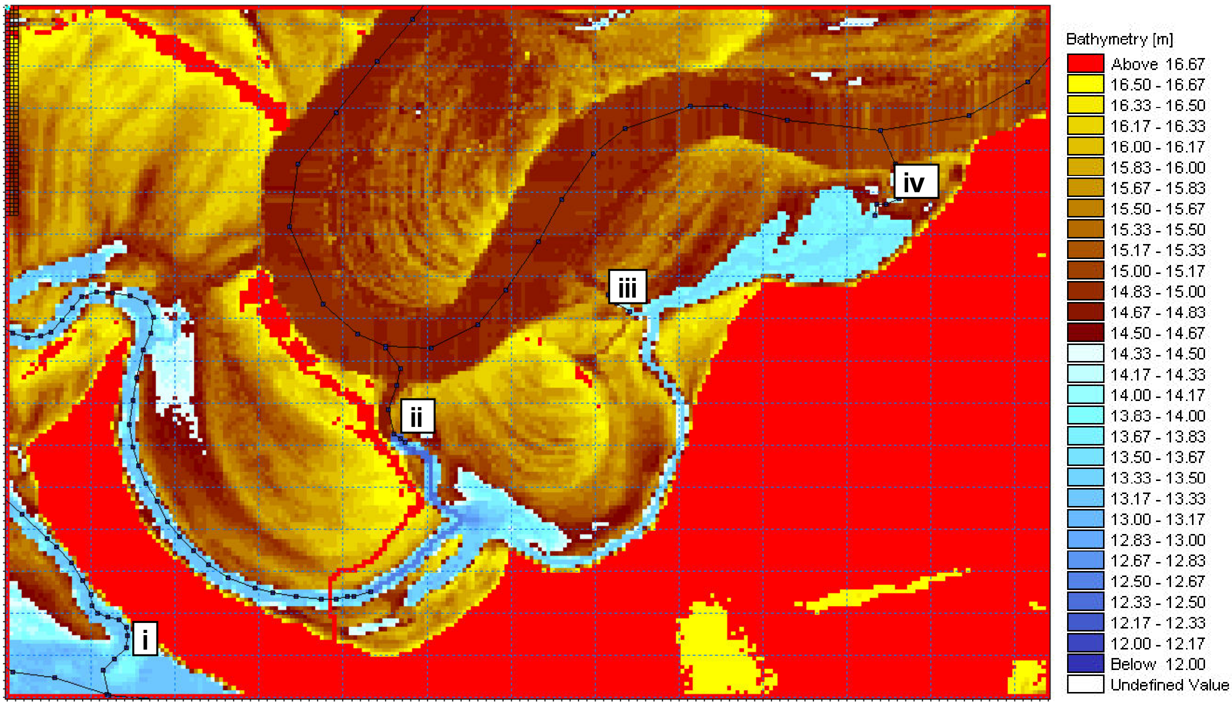

Figure 48: Basic modelling setup, linking 2-D topographic map with 1-D river branches.

8.2 Results and Discussion

Scenario 6A – Existing Conditions

The inundation map for the current operating regime is shown in Figure 49. The inundated area lying between Banks F and F1 is shown to be generally shallow (i.e. approximate maximum depth of 0.5 m), which makes the area conducive to wader and water fowl habitat. In this control scheme, flows are negligible through Banks D and F1, and approximately 20 ML/d entering through Bank F1.

Velocities are generally negligible to slow, as indicated in Figure 50. This is expected given that the water level in this area under existing conditions is strongly influenced by Lock 4 upper pool level. A length of Tanyaca creek between Banks D and F demonstrate elevated velocities, with a number of isolated spot velocities even reaching the acceptable fish passage range (i.e. 0.18 to 1.4 m/s). These specific velocity results should be taken with care however, given that bathymetry elevations were estimated in this area due to the missing data in typically inundated portions of the creek.

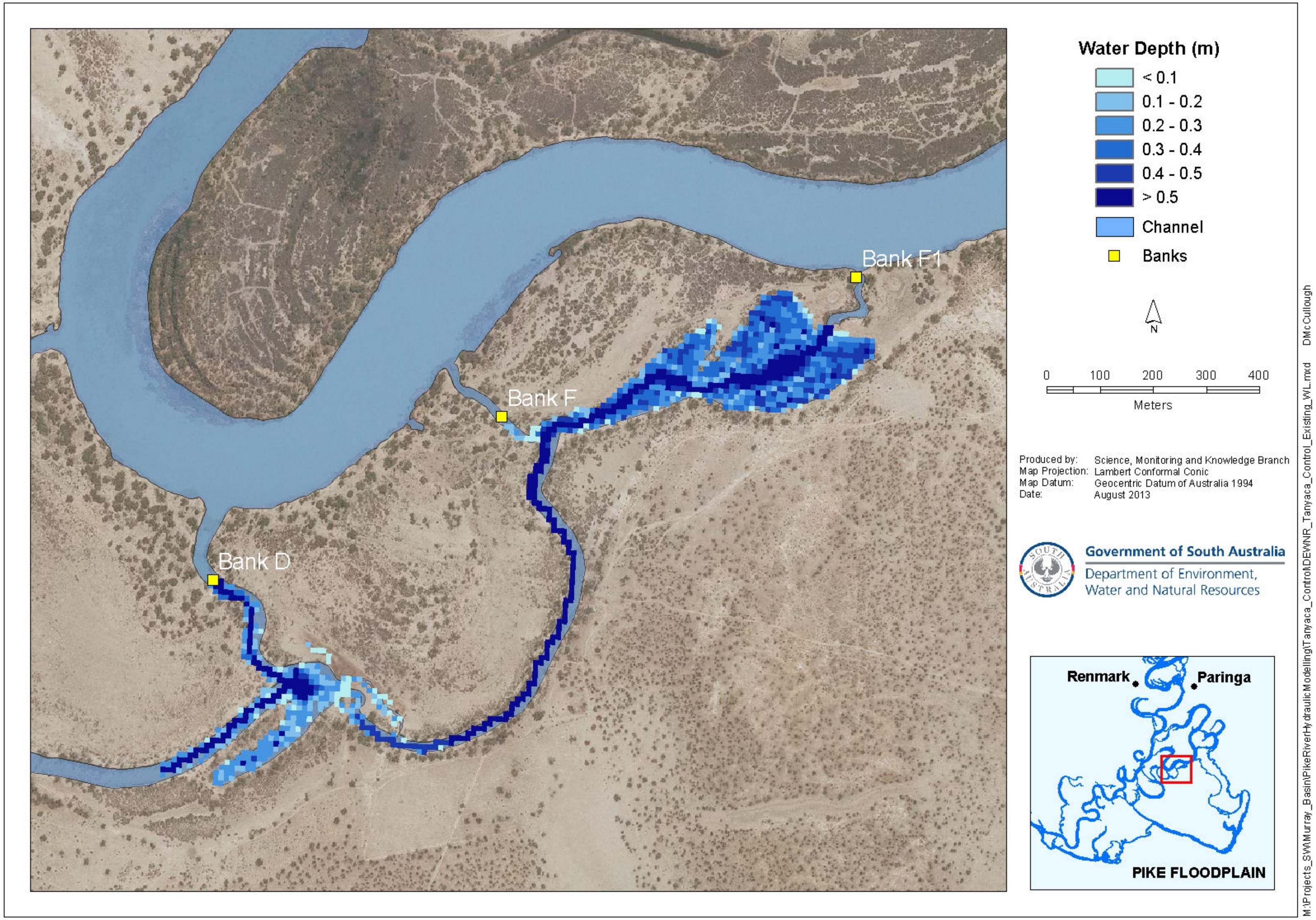

Figure 49: Inundation extent under current (existing) bank configuration.

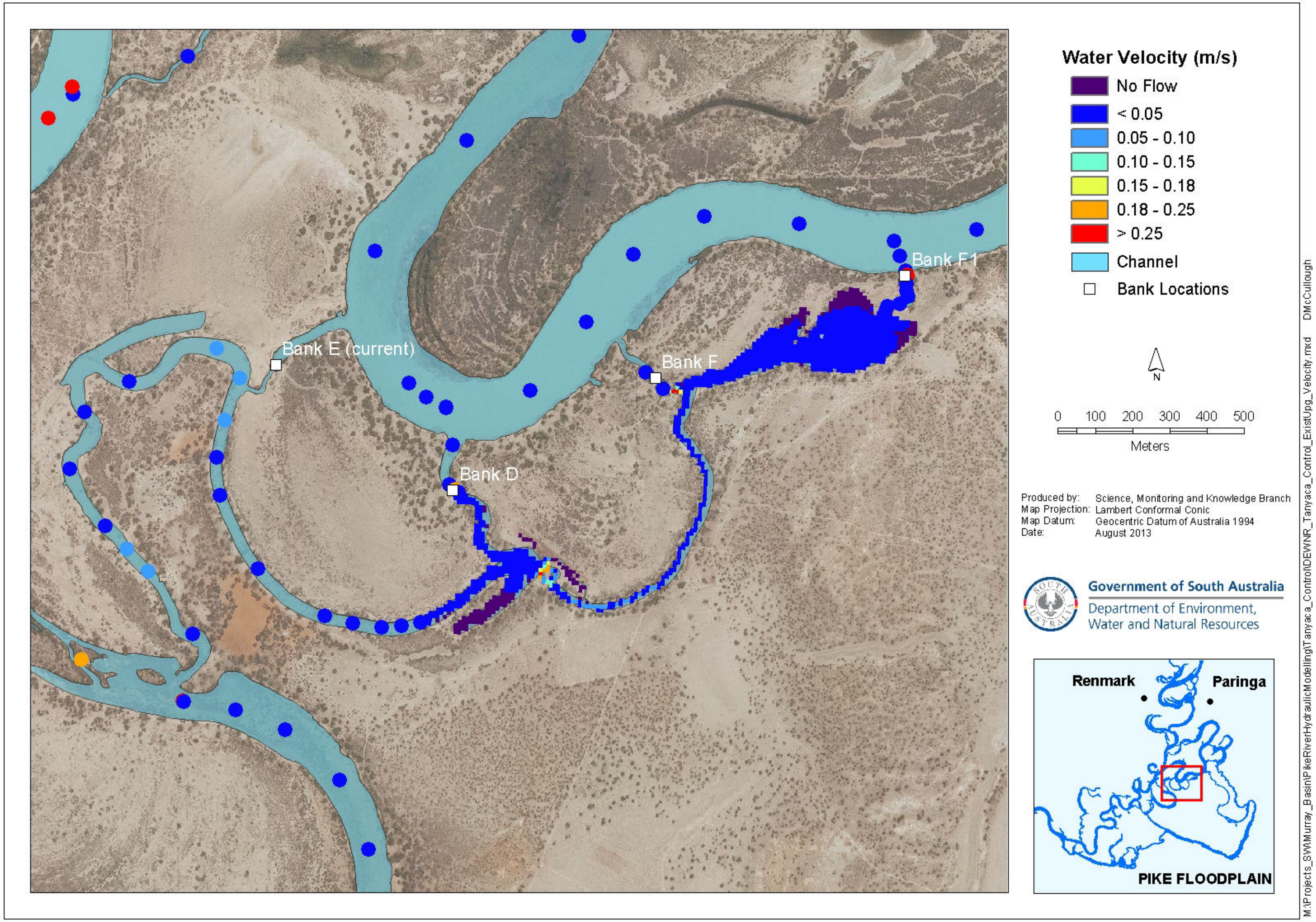

Figure 50: Velocities under the current (existing) bank configuration.

Scenario 6B – Upgraded Banks, 500 ML/d through Bank D

Inundation extent for the case of upgraded Banks D, F and F1, where flow through Bank D is set to 500 ML/d (i.e. no flow through Banks F and F1), is shown in Figure 51. Both the inundation extent and water depths directly below Bank D (area 'A' in Figure 51) are shown to increase noticeably under the elevated flow through Bank D, locally raising the creek's depth to above 0.5 m, and causing some break out onto the adjacent low lying area.

The inundation extent between Banks F and F1 (area 'B' in Figure 51) is seen to correspond closely to that of the existing scenario (Scenario 6A). The depths in this area are seen to be marginally deeper than for the existing case, however remain reasonably shallow (i.e. approximate maximum depth of 0.6 m), indicating that the wader and water fowl habitat may not necessarily be adversely affected.

Velocities generally fall within the ideal fish passage range (0.18 to 1.4 m/s) below Bank D in Tanyaca Creek. Much of the area downstream of Banks F and F1 is in a non-flowing state, indicating that the elevated flow through Bank D is creating a backwater effect in this area given the lack of flow through Banks F and F1.

Scenario 6C – Upgraded Banks, 450 ML/d through Bank D & 50 ML/d through Bank F1

Scenario 6C represents the same bank configuration to that in the previous scenario, with an alteration in flow distribution to 450 ML/d at Bank D and 50 ML/d at Bank F1. The inundation extent in this case (refer to Figure 53) is similar to that of the previous scenario, however water depths between Banks F and F1 (area 'B') in particular are generally raised to depths exceeding 0.5 m (i.e. approximate maximum depth of 0.8 m). This difference demonstrates the effect that operating Bank F1 in its upgraded configuration may have on the downstream wader and water fowl habitat. The velocity map in Figure 54 shows that velocities downstream of Bank D and through Tanyaca Creek remain acceptable for fish passage as for the previous scenario. In contrast, velocities in the area of creek between Banks F and F1 are generally slow (< 0.05 m/s) under the additional flow from Bank F1, rather than in a state of no flow as in the previous scenario.

Given the increase in water levels and slow velocities simulated for this scenario, under normal operation in the upgraded case it may be desired to set flow through Bank F1 to little or no flow, and only operate the structure under specific situations e.g. temporary flushing of the area.

As mentioned previously, velocities and depths may be affected by estimations of bathymetric elevations, however the results indicate that under the upgraded bank configurations water velocities should be acceptable for fish passage when elevated flows through Bank D in particular can be maintained.

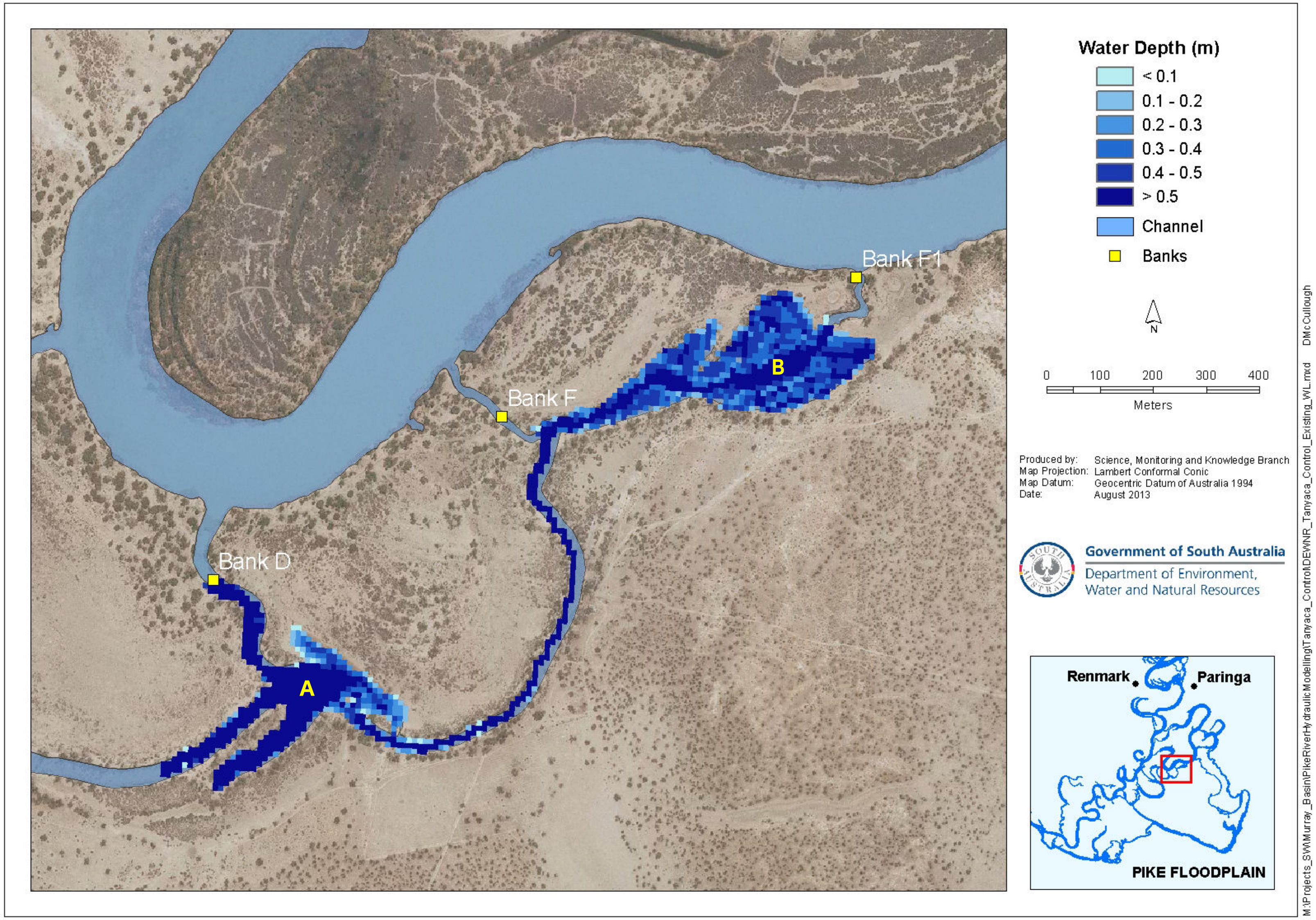

Figure 51: Inundation extent downstream of Banks D, F and F1, 500 ML/d at Bank D. 'A' and 'B' indicate inundated areas of interest.

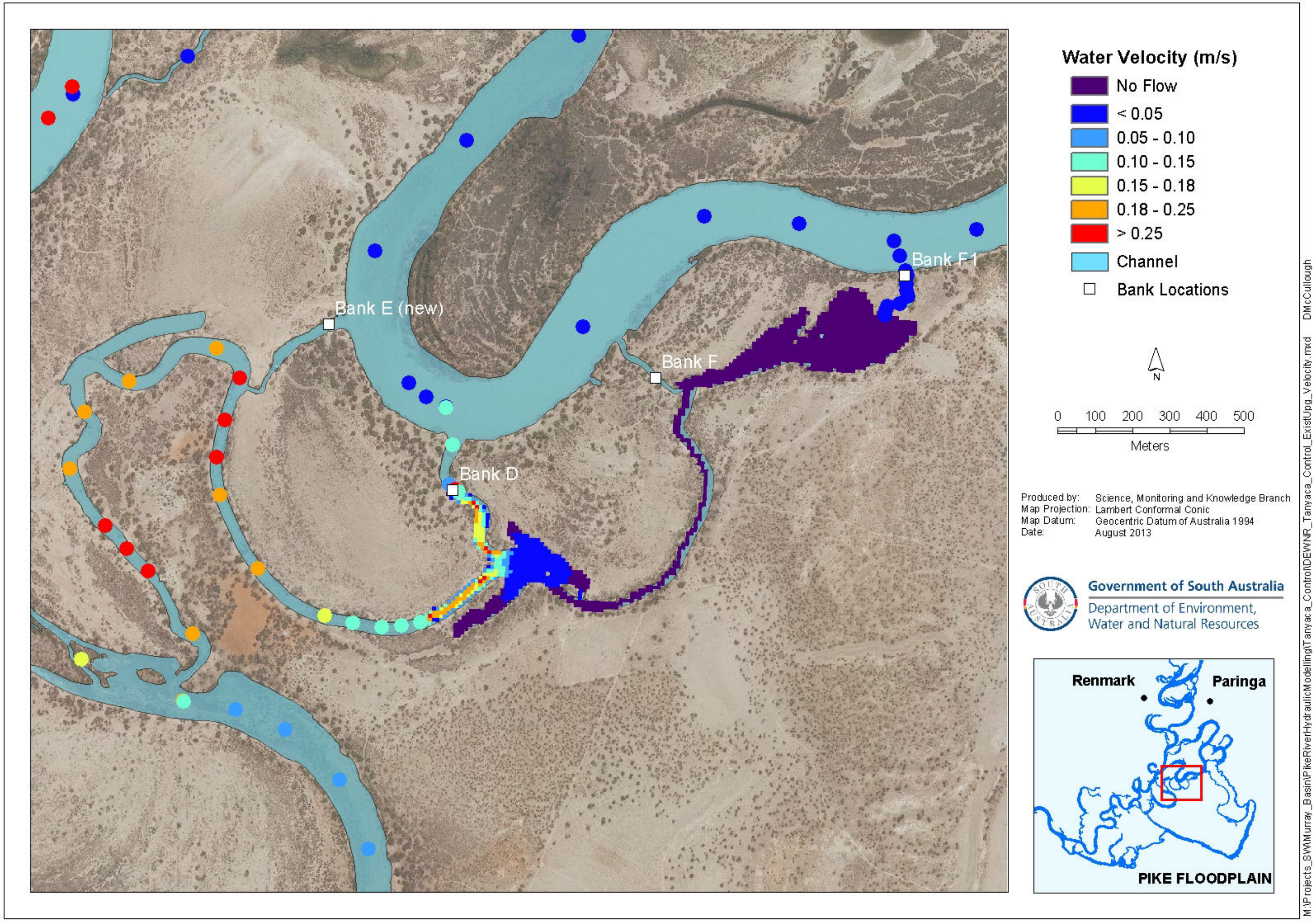

Figure 52: Velocities under upgraded Banks D, F and F1, 500 ML/d at Bank D.

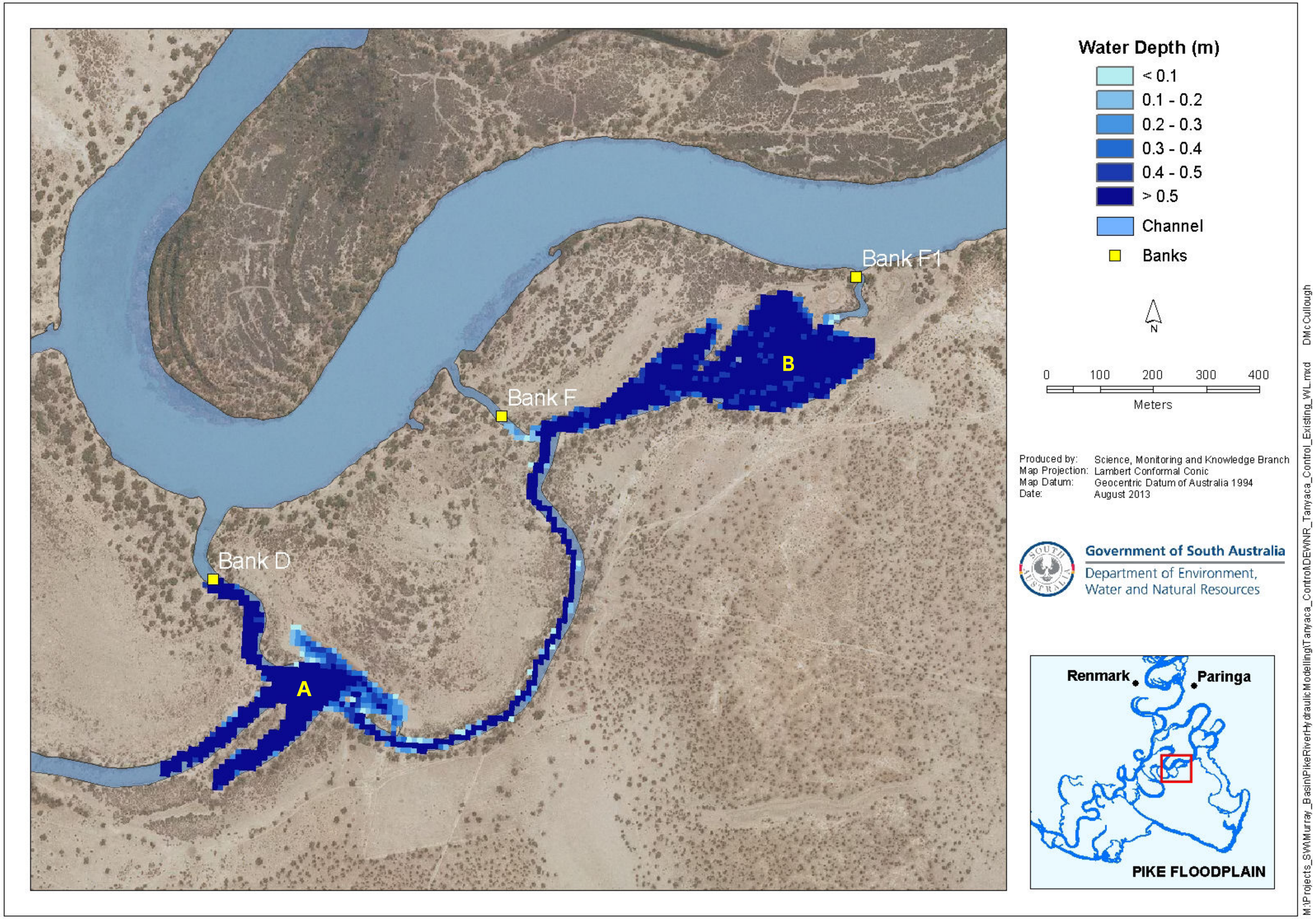

Figure 53: Inundation extent downstream of Banks D, F & F1, 450 ML/d at Bank D and 50 ML/d at Bank F1. 'A' and 'B' indicate inundated areas of interest.

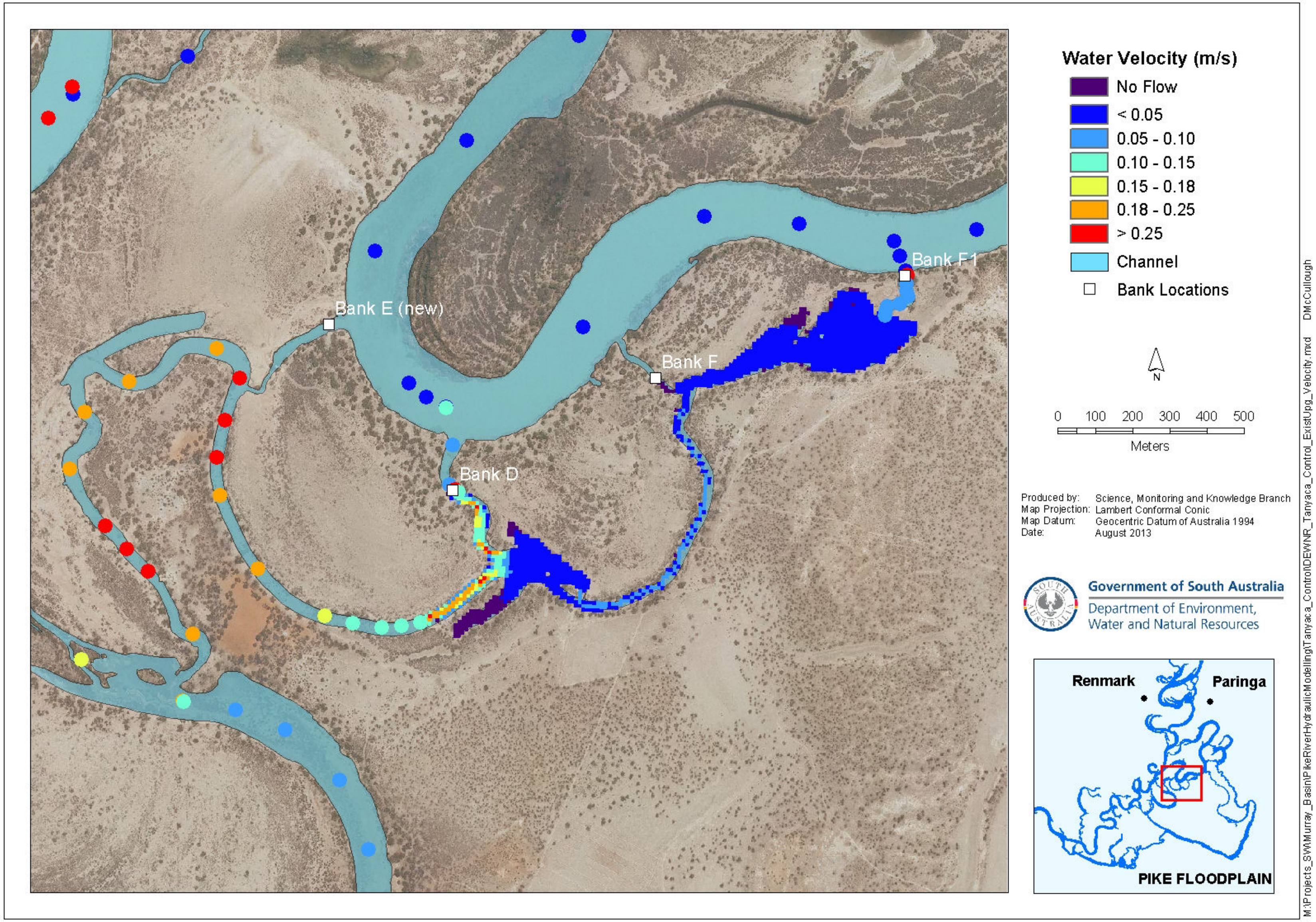

Figure 54: Velocities under upgraded Banks D, F and F1, 450 ML/d at Bank D & 50 ML/d at Bank F1.

Scenario 6D – Proposed Tanyaca Environmental Regulator as alternative control at 500 ML/d

Using the proposed Tanyaca Creek regulator as an alternative control arrangement (Banks D, F and F1 removed) provides the greatest inundation extent of the four scenarios tested, as shown in Figure 55. Flows of ~ 447 ML/d at the current Bank D site and 26-27 ML/d through each of F and F1 sites are achieved with the regulator set point flow at 500 ML/d. The majority of the area inundated between Banks F and F1 exceeds 0.5 m depth (approximate maximum depth of 1.3 m), with shallow areas (< 0.5 m depth) occurring within low lying areas where water has spread under the elevated water level conditions.

Observing close ups of the main areas of interest as indicated by locations 'A' and 'B' in Figure 55, additional inundation has mainly occurred to the east of location 'A' (Figure 56), while inundation extent has extended to the north of location 'B' (Figure 57), compared to current inundation extent. It should be noted that the areas where inundation has extended represent sparsely vegetated areas based on the satellite imagery, indicating that there will be very little impact on existing red gums downstream of current Banks D, F and F1. Exact numbers of red gums affected cannot be elucidated without further detailed survey work on vegetation and topography in the area.

Water velocities downstream in Tanyaca Creek correspond to those in the previous upgraded bank scenarios as shown in Figure 58. Upstream of the proposed regulator site velocities are reduced to below that of ideal fish passage velocities (0.18 to 1.4 m/s), which is a result of the greater cross-section of flow causing a reduction in mean velocity for a given flow rate. Note however that a significant portion of the creek below the current Bank D site exceeds 0.10 m/s, which may remain sufficient for fish passage given the acceptable velocities below the Environmental Regulator.

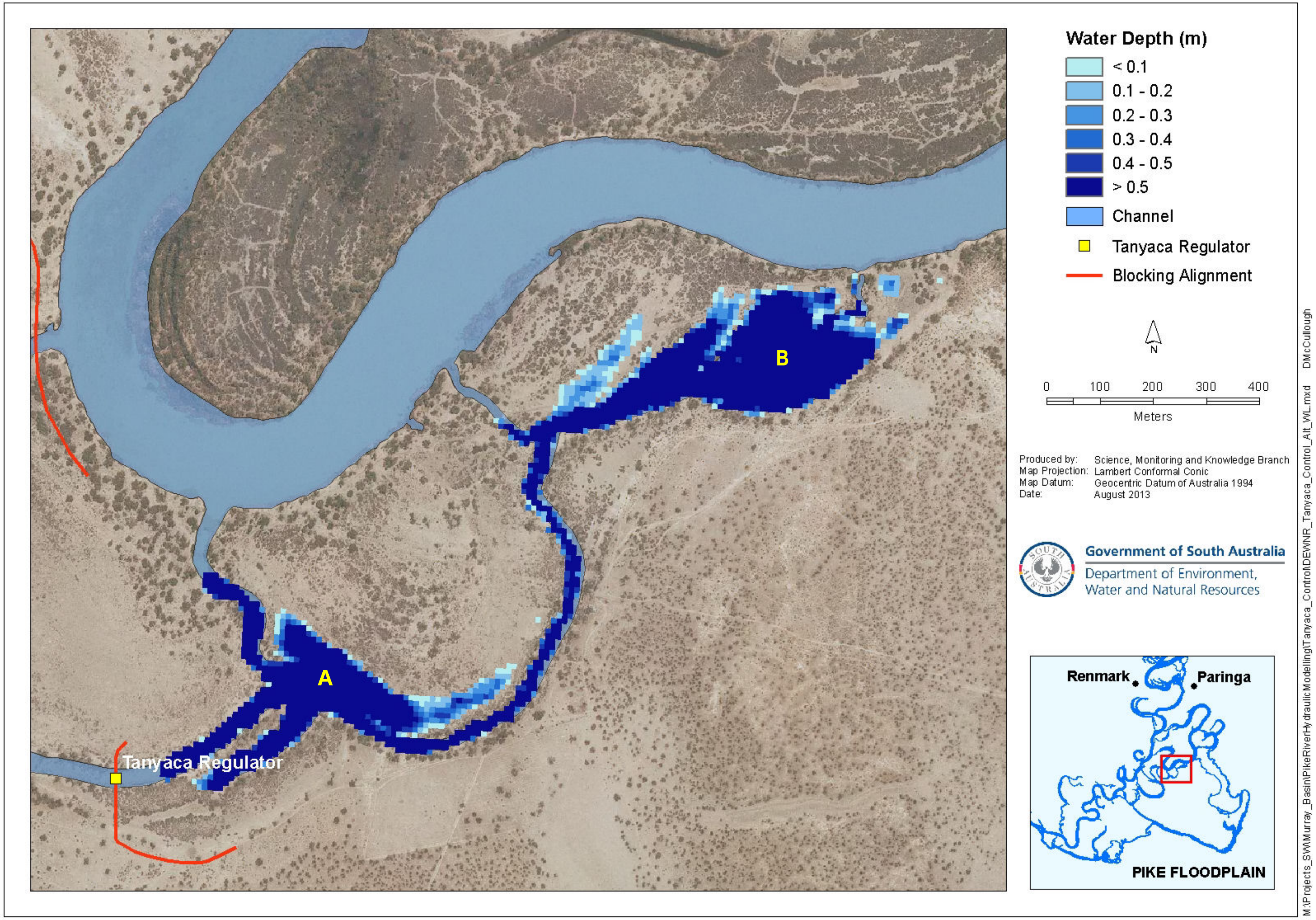

Figure 55: Inundation extent upstream of proposed Tanyaca Creek Environmental Regulator under the alternative control scheme, passing 500 ML/d.

Figure 56: Inundation extent in area A from Figure 55.

Figure 57: Inundation extent in area B from Figure 55.

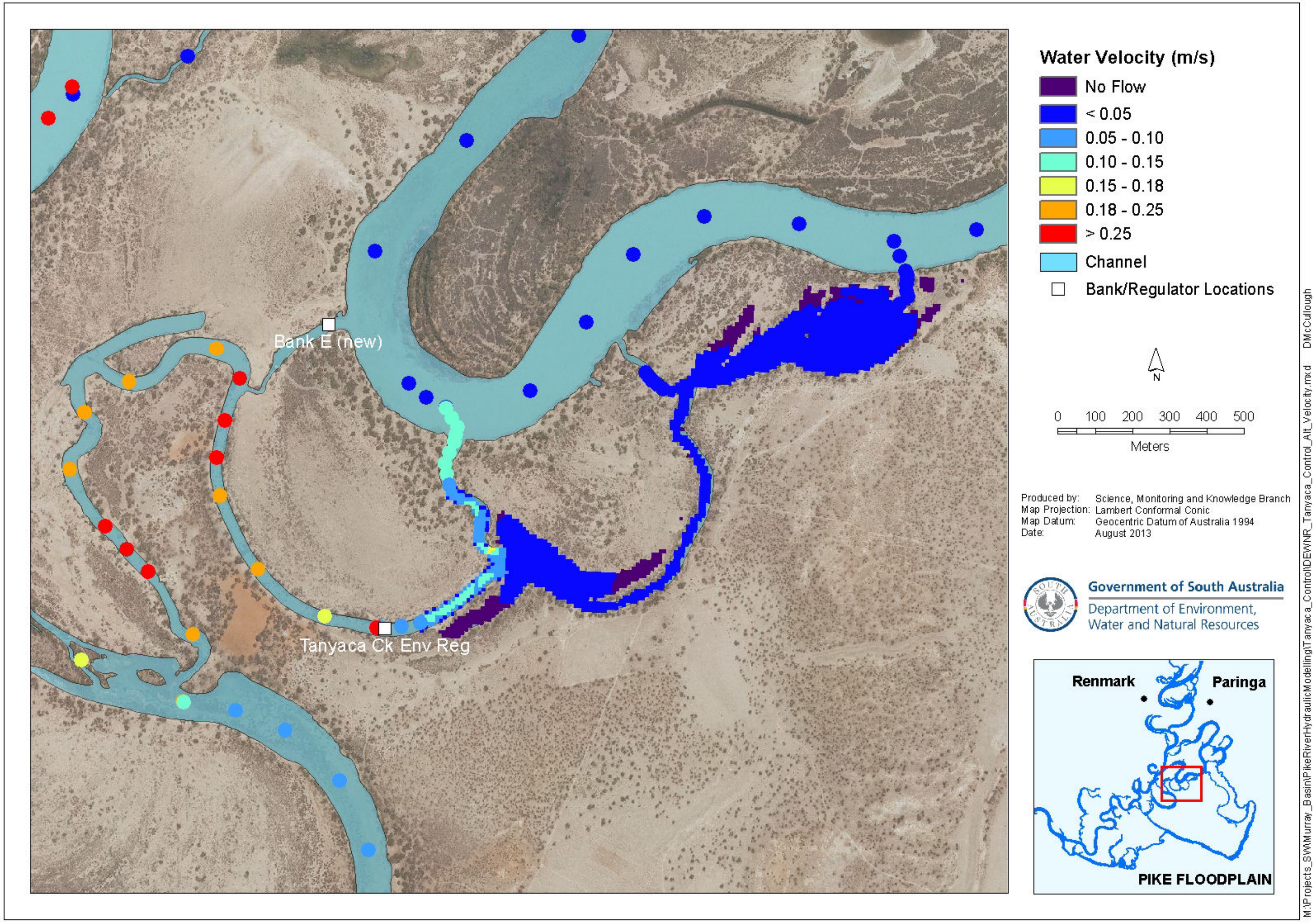

Figure 58: Water velocities under the alternative control scheme, with the proposed Tanyaca Environmental Regulator passing 500 ML/d.

Scenario Comparison

Comparison of inundation results to that under existing conditions indicates that inundation extent increases marginally under both upgraded bank scenarios (i.e. Scenarios 6B and 6C), and increases significantly under the alternative control scenario (Scenario 6D). Referring to Table 11, flow through upgraded Bank D (Scenario 6B) results in an increase in inundated extent of only 1.5 ha, while flow through Bank D and F1 (Scenario 6C) leads to a slightly larger increase of 1.7 ha. Under the alternative control scenario (6D), inundated area increases by over 8 ha from existing conditions with Banks D, F and F1 removed.

Average depth of inundation increases correspondingly to increases in inundated area (Table 11). Upgraded conditions show an increase in average depth of inundation of 0.2 m when only Bank D is flowing, and 0.3 m when flow is occurring through both Banks D and F1. The alternative control scheme using the Tanyaca regulator shows an increase in depth approximately double that of the upgraded conditions, at +0.6 m over existing conditions. These results are reflected in the total volume of inundation (Table 11), with increases of 34 ML, 48 ML and almost 150 ML for Scenarios 6B, 6C and 6D, respectively, over existing conditions. It should be noted that inundated area, depth and volume calculations may be affected by estimations of bathymetry where DEM data is incomplete, and further survey work is required to provide a greater confidence in the analysis.

Table 11: Difference in inundated area and volume for each scenario tested.

Inundation Parameter	Change in Inundation Extent Compared to Existing Conditions		
	Scenario 6B	Scenario 6C	Scenario 6D
Difference in Area* (ha)	+1.5	+1.7	+8.2
Difference in Average Depth* (m)	+0.2	+0.3	+0.6
Difference in Volume* (ML)	+34	+48	+148

* Area, depth and volume may be affected by estimated depths due to missing data in DEM.

A breakdown of inundated area by depth for each scenario is shown in Figure 59. Depths under existing conditions are distributed relatively evenly across the various depth categories. Under 'upgraded' and 'alternative' control regimes, depths are weighted towards the higher end of the scale, with the majority of inundated area lying at > 0.5 m depth. Note that the depth distribution under Scenario 6B yields the closest representation of existing conditions of the scenarios tested, with a comparative decrease in inundated area less than 0.4 m depth and an increase in inundated area exceeding 0.4 m depth.

Comparison between the two upgraded bank scenarios (6B and 6C) indicates that allowing flow through Bank F1 skews the depth distribution towards greater depths of inundation. This suggests that the relative flows between Banks D and F1 can be 'fine-tuned' such that extent of inundation in the area between Banks F and F1 can be controlled.

While Scenario 6D shows the greatest inundated area exceeding 0.5 m depth of the four scenarios tested, it also shows that inundated area is distributed relatively evenly in depth categories under 0.5 m. In particular, depths under 0.2 m represent greater areas of inundation than even the existing scenario. This may indicate that suitable wader and water fowl habitat may be available under the alternative control scenario, although further ecological analysis will be required to fully assess its suitability.

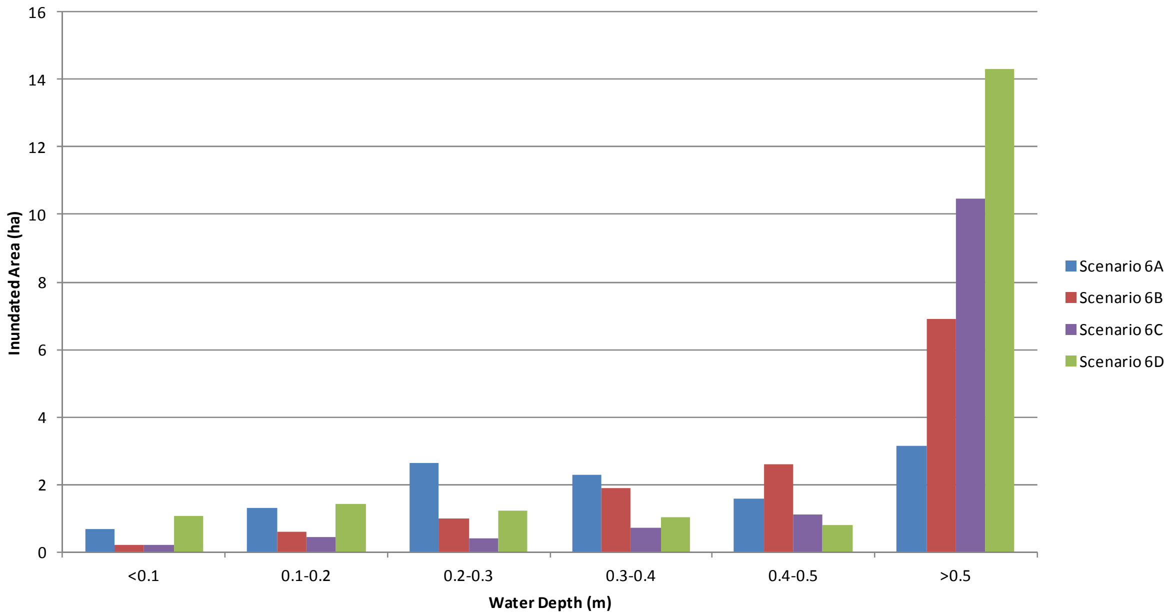

Figure 59: Inundated area by water depth for each scenario tested.

Velocity distributions based on approximate reach length for each scenario tested are shown in Table 12 and graphically in Figure 60. Note that reach lengths were measured manually within ArcGIS, with lengths in the 1-D section of the model being interpolated between adjacent reporting locations. These results can therefore be viewed as possessing a significant degree of error (i.e. a 95% confidence interval of approximately ± 50m based on the differences in total length of creek measured in each case), and care should be taken when using these results for further analysis.

Table 12: Water velocity based on approximate reach length for Scenario 6.

Scenario	Reach length (m) per velocity range (m/s)						
	No velocity	0 – 0.05	0.05 – 0.10	0.10 – 0.15	0.15 – 0.18	0.18 – 0.25	> 0.25
6A	0	4475	843	25	20	10	20
6B	1936	135	90	344	301	1792	750
6C	207	967	949	365	232	1812	747
6D	0	2009	345	395	207	1547	785

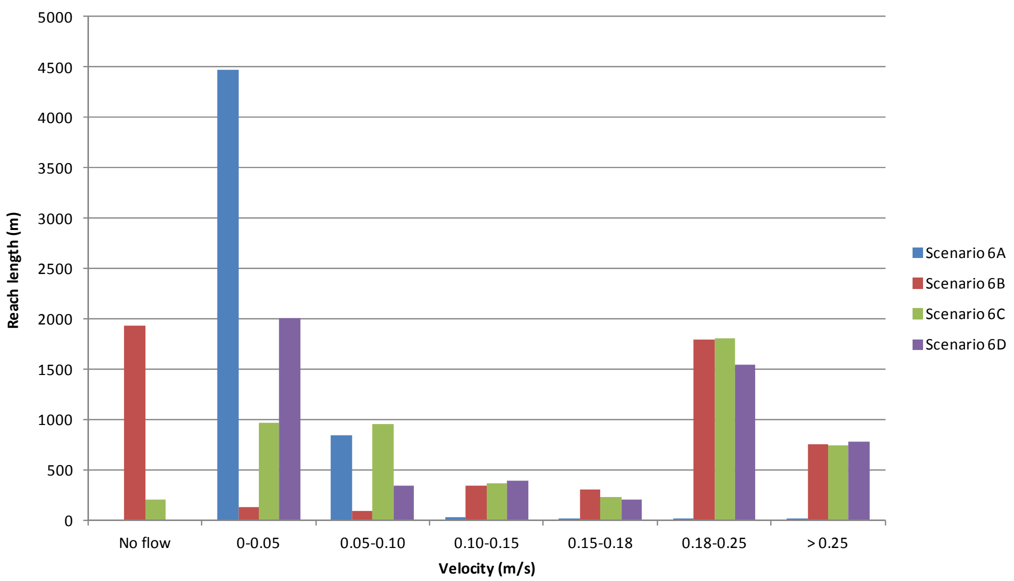

Figure 60: Velocity distribution by total reach length.

Based on the results, existing conditions (Scenario 6A) are shown to provide the least favourable velocities for fish passage (i.e. ideal fish passage range of 0.18 to 1.4 m/s), with only isolated areas exceeding 0.18 m/s, while the majority of creek length is present under 0.10 m/s.

Scenarios 6B and 6C show similar velocity distributions within the ideal fish passage range, mainly driven by similar velocity patterns downstream of the proposed Tanyaca regulator site (i.e. 500 ML/d passes through this reach in each case under similar water levels). A significantly greater length of reach is present under 'no flow' conditions in Scenario 6B however, predominantly located in the reach between Banks F and F1. In either case however, the Bank D site represents the preferred fish passage path.

Scenario 6D has approximately the same length of creek exceeding 0.25 m/s as Scenarios 6B and 6C, which again can be attributed to the same flow rate in the reach downstream of the proposed Tanyaca regulator site. There is however noticeably less reach length between 0.18 and 0.25 m/s (compared to that for Scenarios 6B and 6C) owing to the rise in water level (i.e. velocity decreases as water level increases for a given flow rate). The rise in water level also contributes to an increase in length of creek containing velocities under 0.05 m/s and a reduction in creek length containing velocities between 0.05 to 0.10 m/s compared to Scenarios 6B and 6C. Note that despite the differences in water depth above the proposed Tanyaca regulator site for Scenarios 6B to 6D, the water levels downstream of this site are comparable at approximately 14.12 m AHD given that 500 ML/d flows through this reach in each case.

8.3 Summary

Analysis of four separate control scenarios representing existing, upgraded and alternative control methodologies suggests that using the upgraded or alternative control regimes results in an increase in the total inundated volume (compared to existing conditions) in the area between Banks D, F, F1 and the proposed Tanyaca Regulator. The alternative control scenario was observed to yield the greatest increase in extent of inundation of the three scenarios investigated. Note however that the analysis is expected to be affected by estimates of bathymetry due to missing data, and further survey work is strongly recommended in order to increase the confidence of the analysis.

Impact on existing native vegetation is not expected to be significant in any of the upgraded or alternative control scenarios based on the analysis, and fish passage conditions are expected to be improved in these control scenarios over existing conditions. The increase in inundated depth for upgraded and alternative control methodologies is expected to adversely impact on wader and water fowl habitat directly downstream of current Bank F and F1 sites, although additional shallow (<0.5 m) areas inundated under the alternative control scheme in particular may continue to allow adequate habitat for these bird guilds. Further ecological analysis and vegetation survey work is required to confirm these conclusions, however it is suggested that the increase in velocity associated with preferred fish habitat and the increase in inundated area will better represent 'natural' hydrological conditions of the floodplain, and if so will be preferred over existing conditions in any case.

9. Scenario 7 – Variation in Flows and Water Levels at Upgraded Structures in the Pike Anabranch Complex

The following simulation examines the effect of various combinations of River Murray flow and Pike Anabranch Complex inflows (through Deep Creek, Margaret Dowling, and/or Banks B and C) on the flows and water levels at proposed upgraded structures within the anabranch, with a specific focus on Bank D downstream water level. Additionally, the River Murray flow at which the maximum height of Bank D is exceeded (at 15 m AHD) is also estimated. Results from the simulation are intended to be used to inform engineering designs for the structures.

9.1 Model Simulation

Simulations were performed for the scenarios identified in Table 13. Scenarios 7A and 7B were simulated using the MIKE FLOOD model, allowing flow outside channels to be accounted for. In each of these cases the model was configured with upgraded structures at Banks B, C, D and F1 fully open, 1,000 ML/d total inflow through Margaret Dowling (400 ML/d) and Deep Creek (600 ML/d), while boundary conditions were set at 50,000 ML/d flow at Lock 5 and 13.85 m AHD upstream of Lock 4 for 7A, and 40,000 ML/d and 13.35 m AHD for 7B (based on River Murray backwater curves and monitored data).

Scenarios 7C to 7G were analysed using the 1-D model given in-channel flows at these river flows and the rapid simulation completion times possible with this model version. In each case the model was configured with Banks B, C and F1 fully closed, Bank D configured with between 0 and 3 culverts open and other flows and boundary conditions altered depending on the scenario tested (refer to Table 13). Relative inflows through Margaret Dowling and Deep Creek were set based on a 40-60% split of the total desired inflow. Total flow down Pike River was controlled to 350 ML/d for each scenario by altering the blocking height (i.e. stop log or gate height) at Bank D using a trial-and-error solution.

Table 13: Scenarios for Analysis of Tailwater Level at Upgraded Bank D.

Scenario	Flow at Lock 5 ML/d	Lock 4 Upper Pool Level m AHD	Inflow* ML/d	No. Bank D culverts open	Bank B/C/F1 Open?
7A	50,000	13.85	1,000	8 (maximum)	Yes
7B	40,000	13.35	1,000	8 (maximum)	Yes
7C	10,000	13.20	1,000	3	No
7D	10,000	13.20	750	2	No
7E	10,000	13.20	550	1	No
7F	10,000	13.20	350	0 (fully closed)	No
7G	5,000	13.20	350	0 (fully closed)	No

* Combined flow through Margaret Dowling and Deep Creek.

The 1-D model required a minor recalibration to address an issue identified with Mundic Lagoon water level, which was found to be reporting a level approximately 0.2 m below the normal operating water level (i.e. ~14.75 m AHD), despite operating with Bank D fully closed.

Manning's roughness values of Mundic Lagoon outlet streams were increased from the original value of 0.028 to values in the order of 0.050 to 0.065 in order to better approximate the actual water levels expected in Mundic Lagoon and Pike River, using monitoring data from the State Surface Water Database as an additional calibration guide.

Data produced from the simulation results for each scenario include:

- Flows and water levels upstream and downstream of structures at Banks B, C, D, F1 and Coombs Bridge, plus total flow through Pike River; and
- The required stop-log height at Bank D (where applicable) to ensure a minimum total flow 350 ML/d passes through Pike River.

Additionally, the flow at which the maximum height of Bank D is exceeded is also identified, assuming an obvert level of 15 m AHD for the proposed structure.

9.2 Results and Discussion

Simulation results are presented in Table 14. The results indicate that flow through Bank D at 50,000 ML/d at Lock 5 (Scenario 7A) reaches approximately 1,800 ML/d, resulting in a downstream water level at Bank D of 15.55 m AHD. The downstream level at Bank D under a Lock 5 flow of 5,000 ML/d and Bank D fully closed was found to be 13.24 m AHD (Scenario 7G), resulting in a head of approximately 1.5 m across the structure.

Flows into Pike River were successfully controlled to a minimum of 350 ML/d by manipulating the blocking height at Bank D (Scenarios 7C to 7E). A required blocking height from the invert level (i.e. 12.2 m AHD) was found to be between 1.60 to 1.65 m for the scenarios tested.

A flow of 40,000 ML/d at Lock 5 is shown to yield a Mundic Lagoon water level (i.e. upstream Bank D) of almost 15 m AHD, suggesting that river flows exceeding this value will result in an overtopping of Bank D (N.B. Bank D will need to be fully opened when water level exceeds the obvert height of the structure). Conversely, as flows drop below 40,000 ML/d at Lock 5, stop logs will need to be reinstated at Bank D to avoid excessively draining Mundic Lagoon.

Table 14: Flow, Upstream and Downstream Water Levels for various structures within the Pike Anabranch Complex.

Scenario	Blockage Height	Bank D			Bank F1		
	Bank D	Flow	U/S WL	D/S WL	Flow	U/S WL	D/S WL
	m	ML/d	m	m	ML/d	m	m
7A	Fully open	1812	15.57	15.55	86	15.57	15.56
7B	Fully open	1649	14.98	14.96	101	15.00	14.95
7C	1.60	650	14.72	14.38	0	-	-
7D	1.65	400	14.72	14.16	0	-	-
7E	1.65	200	14.72	13.92	0	-	-
7F	Closed	0	14.72	13.33	0	-	-
7G	Closed	0	14.72	13.24	0	-	-

Scenario (cont.)	Bank B			Bank C			Coombs Bridge			Total Pike River
	Flow	U/S WL	D/S WL	Flow	U/S WL	D/S WL	Flow	U/S WL	D/S WL	Flow
	ML/d	m	m	ML/d	m	m	ML/d	m	m	ML/d
7A	1748	15.66	15.63	1692	15.65	15.61	1424	15.53	15.51	1623
7B	688	15.07	15.06	947	15.07	15.03	853	14.96	14.95	885
7C	-	-	-	-	-	-	350	14.68	14.67	350
7D	-	-	-	-	-	-	350	14.68	14.67	350
7E	-	-	-	-	-	-	350	14.68	14.67	350
7F	-	-	-	-	-	-	350	14.68	14.67	350
7G	-	-	-	-	-	-	350	14.68	14.67	350

10. Scenario 8 – Flow Through Alternative Bank D Arrangements at 50,000 ML/d Flow at Lock 5

The following simulations examine the maximum flow achievable through alternative Bank D culvert designs, with comparison made to the current design option of 8 no. 2100 x 2100 mm box culverts. The alternative designs reduce the number of box culverts and increase the size of each culvert from the current arrangement, while also increasing the length of the spillway. Results from the simulations are used to inform engineering design works.

10.1 Model Simulation

The same model configuration used for Scenario 7A was used as the basis for testing the alternative Bank D design scenarios. Table 15 shows the alternative Bank D arrangements analysed, with comparison to the configuration of Scenario 7A (refer to the preceding chapter for details).

One of the design aims was to maximise the length of the spillway, which was calculated using the detailed plan drawing of the current arrangement (i.e. 8 no. 2100 x 2100 mm box culverts). The current arrangement indicates a total length of 28.72 m available for the Bank D structure (including 10 m of spillway). The spillway lengths of the alternative arrangements were hence calculated using the following formula:

$$\text{Spillway length (metres)} = 28.720 - 3 \times n$$

where n is the number of box culverts in the configuration. Refer to Table 15 for calculated spillway length for each scenario.

Table 15: Scenarios for Analysis of Flow and Water Level under Alternative Bank D Configurations.

Scenario	No. Bank D culverts	Culvert size mm x mm	Length of spillway, m
7A	8	2100 x 2100	10.00
8A	2	2700 x 2700	22.72
8B	3	2700 x 2700	19.72
8C	4	2700 x 2700	16.72

For each scenario:

- Flow at Lock 5 is set at 50,000 ML/d
- Lock 4 upper pool level is set at 13.85 m AHD
- Combined inflow through Margaret Dowling and Deep Creek is 1,000 ML/d
- Culverts at Banks B, C, D and F1 are fully open

The main variations in the model configuration between each scenario included the number and dimensions of the box culverts and the weir (i.e. road) profile above the box culverts.

The weir portion of Bank D includes a road base directly above the culverts at reference level 15.4 m AHD, and a spillway on the right bank (southern) side with reference level of 15 m AHD (including a 4 m transition gradient between road base and spillway).

10.2 Results and Discussion

Table 16 shows the modelled flows and water levels at various locations in the Pike Anabranch Complex under the alternative Bank D arrangements. Also included are the results from Scenario 7A, which indicates the flows and levels under the current arrangement of 8 box culverts.

The results indicate that flows vary at each location while water levels are relatively consistent across the scenarios tested. Maximum flows through Bank D are as follows:

- Scenario 8A – Approximately 1470 ML/d for 2 no. 2700 x 2700 mm box culverts
- Scenario 8B – Approximately 1650 ML/d for 3 no. 2700 x 2700 mm box culverts
- Scenario 8C – Approximately 1770 ML/d for 4 no. 2700 x 2700 mm box culverts

These compare to a maximum flow in the current arrangement (Scenario 7A) of approximately 1810 ML/d. Thus, 2 box culverts provides approximately 80% of the maximum flow under the current arrangement scenario, 3 box culverts provides approximately 90% of the maximum flow, and 4 box culverts provides almost 98% of the maximum flow under the current arrangement. Therefore, the alternate arrangement of 4 no. 2700 x 2700 mm box culverts is approximately equivalent to the current design option of 8 no. 2100 x 2100 mm culverts, but also provides a larger spillway length (i.e. approximately 17 m compared to 10 m currently – refer to Table 15).

Note however that factors in addition to maximum flow should also be taken into account when selecting the final design option for Bank D, in particular the requirement for a minimum of approximately 300-400 ML/d to be supplied to irrigators through Pike River under normal River Murray flow conditions, which may make higher maximum flow rates through Bank D undesirable, especially in the event that gates remain fully open following a high flow event. A smaller number of culverts may therefore be appropriate while still meeting ecological requirements downstream of Bank D.

Table 16: Flow, Upstream and Downstream Water Levels for various structures within the Pike Anabranch Complex for Alternative Bank D Configuration Scenarios.

Scenario	Bank D			Bank F1		
	Flow	U/S WL	D/S WL	Flow	U/S WL	D/S WL
	ML/d	m	m	ML/d	m	m
7A	1812	15.57	15.55	86	15.57	15.56
8A	1472	15.57	15.55	120	15.57	15.56
8B	1648	15.57	15.55	109	15.57	15.56
8C	1771	15.57	15.55	92	15.57	15.56

Scenario (cont.)	Bank B			Bank C			Coombs Bridge			Total Pike River
	Flow	U/S WL	D/S WL	Flow	U/S WL	D/S WL	Flow	U/S WL	D/S WL	Flow
	ML/d	m	m	ML/d	m	m	ML/d	m	m	ML/d
7A	1748	15.66	15.63	1692	15.65	15.61	1424	15.53	15.51	1623
8A	1741	15.66	15.63	1636	15.65	15.61	1442	15.54	15.51	1644
8B	1757	15.66	15.63	1661	15.65	15.61	1433	15.54	15.51	1635
8C	1767	15.66	15.63	1677	15.65	15.61	1428	15.54	15.51	1629

11. Scenario 9 – Operation of Bank D with Two Culverts and 'Dual Leaf' Gate Designs

The following simulation examines the discharge through Bank D and water levels upstream and downstream of the structure for a number of different operating scenarios to inform engineering design calculations and operational schemes. Analysis is conducted for a fixed design of the Bank D structure consisting of two box culverts and 'Dual Leaf' gates on each culvert for control. This configuration was the chosen design for Bank D given the requirement to maintain supply of at least ~ 300 to 400 ML/d to irrigators through Pike River at all times (i.e. avoiding excessive lowering of Mundic Lagoon water level through uncontrolled flow through Bank D), while also facilitating operation of control gates by boat when the vehicle track to Bank D is inaccessible following flooding and/or high rainfall events.

11.1 Model Simulation

The model configuration used for the previous Scenario 8A was used as the basis for analysis. The following assumptions were used for configuring the model:

- Flow at Lock 5 = 10,000 ML/d
- Flow through Margaret Dowling = 400 ML/d and Deep Creek = 600 ML/d
- Banks B, C and F1 closed.
- Bank D consisting of
    - 2 no. 2700 x 2700 mm box culverts, 9.8 m length.
    - Invert level of 12.3 m AHD.
    - 'Dual leaf' gates operated in an overshot arrangement, with the 'fully open' condition set with a top of gate level of 13.65 m AHD.
    - Spillway length of approximately 20 m at elevation of 15 m AHD.

The scenarios tested are shown in Table 17, with Gate 1 representing the gate closest to the fishway on the left bank side of the stream. Results reported for each scenario include flow through Bank D and water levels directly upstream and downstream of the structure.

Table 17: Bank D operating scenarios for Scenario 9.

Scenario	Elevation, Top of Gate 1 m AHD	Elevation, Top of Gate 2 m AHD
9A	14.55	15.00 (closed)
9B	14.55	14.25
9C	14.55	13.65 (fully open)
9D	14.25	13.65 (fully open)
9E	13.65 (fully open)	13.65 (fully open)

11.2 Results and Discussion

The results for Scenario 9 are shown in Table 18. Scenarios 9A and 9B indicate that the upstream level in Mundic Lagoon exceeds the top height of the culverts, with some flow expected in the spillway section of the bank. Note that the road base directly above the culverts is designed for a level of 15.4 m AHD, which remains greater than the upstream water level in these cases. This effect is due to the increased flow through Margaret Dowling and Deep Creek from current typical flows (e.g. approximately 250 to 350 ML/d total), which results in an increase in flow through Pike River and hence a rise in water level.

In the case where both gates are 'fully open' (Scenario 9E), the maximum flow through Bank D is at approximately 660 ML/d. The level in Mundic Lagoon, at 14.70 m AHD, is marginally below the 'normal' operating level of 14.75 m AHD. Note that these results are expected to change if flow is reduced through Margaret Dowling and Deep Creek and/or allowing flow through Bank F1.

Table 18: Flows and water levels upstream and downstream of Bank D for Scenario 9.

Scenario	Flow through Bank D ML/d	Upstream Water Level m AHD	Downstream Water Level m AHD
9A	280	15.15	14.02
9B	366	15.06	14.12
9C	504	14.90	14.26
9D	550	14.85	14.30
9E	661	14.70	14.39

12. Summary

A number of scenarios have been simulated to inform design and operational decisions for Pike regulators, including:

- Simulating the anabranch complex under pre-regulated conditions to determine the theoretical flow distribution through the various creeks under "natural" conditions for various River Murray flows,
- Simulating a number of managed inundation scenarios using proposed environmental regulating structures,
- Simulating alternative regulator control options to assess the hydrological and ecological impact of each option, and
- Simulation of flows and water levels at specific structures throughout the floodplain to assist engineering design work.

Simulations were performed with the MIKE FLOOD/MIKE 11 software packages using 1D or 1D/2D coupled model versions depending on the type of simulation required.

13. References

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