Upper Hunter Shire Council

ABERDEEN FLOOD STUDY

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JULY 2013

Level 2, 160 Clarence Street
Sydney, NSW, 2000

Tel: 9299 2855
Fax: 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

ABERDEEN FLOOD STUDY

JULY, 2013

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Client Upper Hunter Shire Council	**Client's Representative** Ben Oliver
Authors Richard Dewar, Bowen Hicks	**Prepared by**
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AAD	Annual Average Damages
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ARI	Average Recurrence Interval
BOM	Bureau of Meteorology
CFERP	Community Flood Emergency Response Plan
DNR	Department of Natural Resources
DTM	Digital Terrain Model
FPL	Flood Planning Level
FSL	Full Supply Level of a dam
GIS	Geographic Information System
GL	Gigalitre
GSAM	Generalised Southeast Australian Method
GTSMR	Revised Generalised Tropical Storm Method LGA Local Government Area
m	metre
m^3/s	cubic metres per second
OEH	Office of Environment and Heritage
PMF	Probable Maximum Flood
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software program (hydraulic computer model)
WBNM	Watershed Bounded Network Model (hydrologic computer model)
1D	One Dimensional hydraulic computer model
2D	Two Dimensional hydraulic computer model

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FOREWORD

The NSW State Government's Flood Policy provides a framework to ensure the sustainable use of floodplain environments. The Policy is specifically structured to provide solutions to existing flooding problems in rural and urban areas. In addition, the Policy provides a means of ensuring that any new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the Government through four sequential stages:

1. ***Flood Study***
    - Determine the nature and extent of the flood problem.
2. ***Floodplain Risk Management Study***
    - Evaluates management options for the floodplain in respect of both existing and proposed development.
3. ***Floodplain Risk Management Plan***
    - Involves formal adoption by Council of a plan of management for the floodplain.
4. ***Implementation of the Plan***
    - Construction of flood mitigation works to protect existing development,
    - use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

The Aberdeen Flood Study constitutes the first stage of the management process for the township of Aberdeen and documents the work undertaken and presents outcomes that define flood behaviour for existing catchment conditions. WMAwater (formerly Webb, McKeown & Associates) were commissioned by the Upper Hunter Shire Council to prepare this flood study on behalf of the Upper Hunter Floodplain Management Committee.

WMAwater has prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. This document does not necessarily represent the opinions of the NSW Government or the Office of Environment and Heritage.

EXECUTIVE SUMMARY

The NSW Government's Flood Policy provides for:

- a framework to ensure the sustainable use of floodplain environments,
- solutions to flooding problems,
- a means of ensuring new development is compatible with the flood hazard.

Implementation of the Policy requires a four stage approach, the first of which is preparation of a Flood Study to determine the nature and extent of the flood problem.

The Aberdeen Flood Study was initiated as a result of past flooding of roads and residential areas, notably February 1955, February 1971 and January 1976.

The specific aims of the Aberdeen Flood Study are to:

- define flood behaviour in the Hunter River adjoining Aberdeen,
- prepare flood hazard and flood extent mapping,
- prepare suitable models of the catchment and floodplain for use in subsequent Floodplain Risk Management Studies and Plans.

Description of River System: The Hunter River has a catchment area of approximately 4,000 square kilometres to Aberdeen. The Hunter River then flows past the urban areas of Muswellbrook, Singleton and Maitland and ultimately into the Pacific Ocean through Newcastle Harbour. The total catchment area of the Hunter River to Newcastle is approximately 22,000 square kilometres thus the area to Aberdeen represents only 20% of the total catchment.

The majority of the catchment to Aberdeen is rural farmlands, natural or semi-natural forests or grasslands. There are scattered towns throughout the catchment but these occupy only a small percentage of the catchment area. The most significant feature affecting the hydrology of the catchment to Aberdeen is Glenbawn Dam on the Hunter River which was completed in 1958.

Aberdeen is located immediately downstream of the confluence of the Hunter River and the Pages River. The town comprises approximately 2000 residents with the majority located on high ground. The railway line provides a significant barrier to flow along the Hunter River, immediately prior to its junction with Dart Brook.

The key phases of the Aberdeen Flood Study that have been undertaken are summarised below:

Review all available data, namely:

- reports, photographs, Council records,
- review of the Scone Flood Study and Glenbawn Dam Assessment of Spillway Adequacy,
- review of historical rainfall data,
- a comprehensive photogrammetric survey was undertaken in 2006 to obtain ground levels,

- collection of historical flood data from reports, maps and a questionnaire survey.

Determine Design Flood Approach: A rainfall-runoff approach was adopted due to the absence of a complete long term historical flood record at Aberdeen that might allow a flood frequency analysis to be undertaken. This rainfall-runoff approach involved the setting up of two computer models - a hydrologic model to convert rainfall to runoff and a hydraulic model to convert the runoff to flows, flood levels and velocities.

Calibration to Historical Flood Levels: Following establishment of both models they were then used to simulate data from several historical floods (1984, 1992, 1996, 1998, 2000). This was achieved by ensuring the flows from the hydrologic model and flood levels from the hydraulic model matched those actually recorded.

Determination of Design Flood Levels: Following calibration of the models design rainfall data and temporal patterns from Australian Rainfall and Runoff (1987) were obtained. This data was then input to the hydrologic/hydraulic models to determine design flood behaviour including the calculation of design flood levels. Due to the limited quality and quantity of the calibration data available and in view of the sensitivity analyses, it is estimated that the order of accuracy is up to ±0.5 m for the study area. These orders of accuracy are typical of such studies and can only be improved upon with additional observed rainfall, flow and flood height data to refine the model calibration.

Flood Problem Areas: The study has indicated that floodwaters will overtop the town levee on the southern bank of the Hunter River and inundate a large number of properties and houses. The entire floodplain will be inundated by up to 2 m to 3 m depth of floodwaters in large floods. This will cause considerable damage to agricultural activities, public facilities (roads, bridges, water supply) and other rural floodplain users. The lower parts of the township of Aberdeen will also be inundated causing damages to buildings and vehicles. Throughout the inundated area there will be a significant risk to life.

Outcomes: The main outcomes of this study are as follows:

- full documentation of the methodology and results,
- preparation of flood contour/hazard and extent maps for the study area,
- establishment of models which will form the basis for a subsequent Floodplain Risk Management Study and Plan.

1. INTRODUCTION

The town of Aberdeen (Figure 1) has a population of approximately 2000 and is located on the eastern bank of the Hunter River, approximately midway between Scone to the north (upstream) and Muswellbrook to the south (downstream) within the Upper Hunter Local Government Area (LGA). The catchment area of the Hunter River to Aberdeen is approximately 3,900 km^2 and the town is located at the confluence of the Hunter River (3,100 km^2) and Dart Brook (800 km^2).

The upper catchment has been extensively cleared for agricultural or other rural activities but there still remains a considerable amount of natural vegetation.

Flooding has been a significant factor in the development of the region with the earliest significant flood recorded in June 1820. Subsequently there have been many floods at Aberdeen of varying size. The largest and most well known occurred in February 1955 and caused considerable damage throughout the Hunter Valley from Scone to Newcastle. Since 1955 there have been significant floods in February 1971 and January 1976. Aberdeen is built on a small rise and thus it is largely only the open space area west of the New England Highway and the residential area east of the highway that are inundated. In 1976 a levee was constructed by the then Water Resources Commission to provide protection for the residential area east of the highway. This levee was not tied into high ground (see aerial photograph below) and so may be outflanked in non-overtopping events. It will also be overtopped in floods of the magnitude of February 1955 or greater.

In view of the need to accurately define the nature and extent of the flood problem, the Upper Hunter Shire Council engaged WMAwater (formerly Webb, McKeown & Associates) to undertake a Flood Study of Aberdeen and the immediate floodplain areas.

The primary objectives of this Study are

- to define the flood behaviour of the Hunter River near Aberdeen by quantifying flood levels, velocities and flows for a range of design flood events under existing catchment and floodplain conditions,

- to assess the hydraulic categories and undertake provisional flood hazard mapping (in accordance with the NSW Floodplain Development Manual (2005)),
- to formulate a suitable hydrologic and hydraulic modelling platform that can be used in a subsequent Floodplain Risk Management Study to assess various floodplain management measures.

As directed by Council, the scope of this study is such that:

- the extent of the hydrologic model covers the entire Hunter River catchment and its tributaries draining to Aberdeen (approximately 3900 km^2),
- the hydraulic model (extent shown on Figure 1) incorporates the Dart Brook and Kingdon Ponds to approximately 7 km upstream of Aberdeen, the Pages River to a point approximately 1 km upstream of the Hunter River confluence, and the Hunter River to approximately 2 km upstream of the Pages River confluence. The downstream limit is to the extent of the Upper Hunter Shire Council local government area (approximately 3 km downstream of Aberdeen).

This report details the results and findings of the Flood Study investigations. The key elements include:

- a summary of available flood related data,
- calibration of the hydrologic and hydraulic models,
- definition of the design flood behaviour for existing conditions through the analysis and interpretation of model results.

A glossary of flood related terms is provided in Appendix A.

All measurements of flow are reported in cubic metres per second or m^3/s. This is standard unit of flow measurement in hydrologic and hydraulic models. However some water authorities in NSW refer to flow in units of megalitres per day of Ml/day. The following conversion is provided.

> *1m^3/s = 1,000 litres/second and there are 60x60x24 = 86,400 seconds/day. Therefore 1m^3/s = 86,400/1,000 = 86.4Ml/day. To convert m^3/s to Ml/day multiply by 86.4 or m^3/s x 86.4 = Ml/day.*

2. BACKGROUND

2.1. Catchment Description

The vast majority of the catchment to Aberdeen comprises either natural or semi-natural vegetation or land used for agricultural or other rural activities. Whilst there are several towns in the catchment, these comprise a very small percentage of the total catchment area and their impact on the runoff characteristics affecting flooding at Aberdeen would be negligible.

The most significant hydrologic feature of the upper catchment is Glenbawn Dam which was constructed in 1958. There are other dams in the catchment but these are of much smaller size and have been ignored in the hydrologic analysis. Further details on Glenbawn Dam are provided in Sections 2.2.6 and 2.2.7.

2.2. Previous Studies

A summary of previous relevant investigations undertaken within the study area is provided in the following sections.

2.2.1. Hunter Valley – Effect of Dams on Flood Levels at Muswellbrook, Singleton and Maitland, 1982 (Reference 1)

This study was undertaken to assess the reduction in flood levels as a result of construction of Glenbawn Dam and Glennies Creek Dam. Glenbawn Dam was completed in 1958 and according to the reference, from 1958 to the date of the report (1982) the only outflow was through the twin outlet valves (each 64 m^3/s). Thus all floods were contained within the dam. The catchment within Glenbawn Dam represents 31% of the total catchment to Muswellbrook (33% to Aberdeen) but only 8% and 7% at Singleton and Maitland respectively.

The study determined a set of pre and post dam outflows for the Hunter River at the dam wall and combined this with the flows from the Pages River, Dart Brook and Kingdon Ponds. This produced a peak flow at Muswellbrook which was converted to a flood level using a rating table (relationship between flood level and peak flow).

For the three locations (Muswellbrook, Singleton, Maitland) a tabulation of pre and post Glenbawn Dam flood levels were determined. Unfortunately there is no detail regarding the actual flows or the methodology of calculating the reduction in flows from construction of Glenbawn Dam. At Maitland there was little difference in level for the majority of events with the largest difference being 0.6 m (October 1942 flood). At Singleton there was also little difference in levels except for 1.2 m for the October 1942 and April 1950 events. At Muswellbrook six events were indicated as having a reduction greater than 1 m. The August 1952 event indicated a reduction of 3.6 m.

The reductions in peak levels for the three major events in recent times are shown on Table 1.

Table 1 Reduction in Flood Level as a Result of Construction of Glenbawn Dam (Reference 1)

Town	Catchment Area (km2)	February 1955		February 1971		January 1976	
		Gauge Level (m)	Reduction (m)	Gauge Level (m)	Reduction (m)	Gauge Level (m)	Reduction (m)
Muswellbrook	4,200	11.5	0.4	11.1	0.2	10.8	0.5
Singleton	16,600	14.6	nil	14.2	0.1	12.7	0.3
Maitland	17,500	12.3	nil	11.5	0.2	10.6	0.2

2.2.2. Muswellbrook Flood Study – 1986 (Reference 2)

The town of Muswellbrook is located approximately 10 kilometres downstream of Aberdeen and has a contributing catchment area of approximately 4,200 km^2. This study undertook flood frequency analysis based on the pre and post dam flood record reported in Reference 1. The pre-dam analysis was undertaken using 78 years of record (1907 to 1984) whilst the post dam analysis was undertaken using 43 years of record (1942 to 1984).

A comparison of the design flood levels from the two sets of analysis was made to determine the decrease in flood levels resulting from construction of Glenbawn Dam (Table 2).

Table 2 Comparison of pre and post Glenbawn Dam Gauge Levels and Peak Flows at Muswellbrook (Reference 2)

AEP (%)	Pre - Glenbawn Dam		Post - Glenbawn Dam		
	Gauge Level (m)	Peak Flow (m^3/s)	Gauge Level (m)	Peak Flow (m^3/s)	Flow Reduction (%)
1	11.69	5580	11.47	4780	14
2	11.26	4140	11.14	3780	9
4	10.68	2890	10.63	2840	2
10	9.35	1790	9.35	1790	0
20	7.43	1090	7.43	1090	0
50	4.56	390	4.56	390	0

The above results indicate no reduction in flood level for the 10% AEP and smaller events with the following reductions for larger events:

- 1% AEP reduction of 0.22 m,
- 2% AEP reduction of 0.12 m,
- 4% AEP reduction of 0.05 m.

The trend for larger reductions in flood level with smaller AEP (i.e. a larger flood) is unusual. Typically the greatest reduction would occur in the smaller more frequent events as the flood mitigation component within the dam could “contain” a greater percentage or all of the runoff from a smaller event than a larger event. The base data used for the post dam flood frequency analysis indicates that the peak flows for the smaller events are reduced. However this is not reflected in the results shown in Table 2.

It is considered that the approach adopted to assess the impacts of Glenbawn Dam in this reference is flawed as it compares two different periods of flood record (78 years for pre dam vs 43 years for post dam). To provide a rigorous comparison the identical period of record should be analysed for the pre and post dam cases. However, it is noted that the accompanying Muswellbrook Flood Inundation Map - 1984 (refer Diagram 1) indicates a completely different set of results.

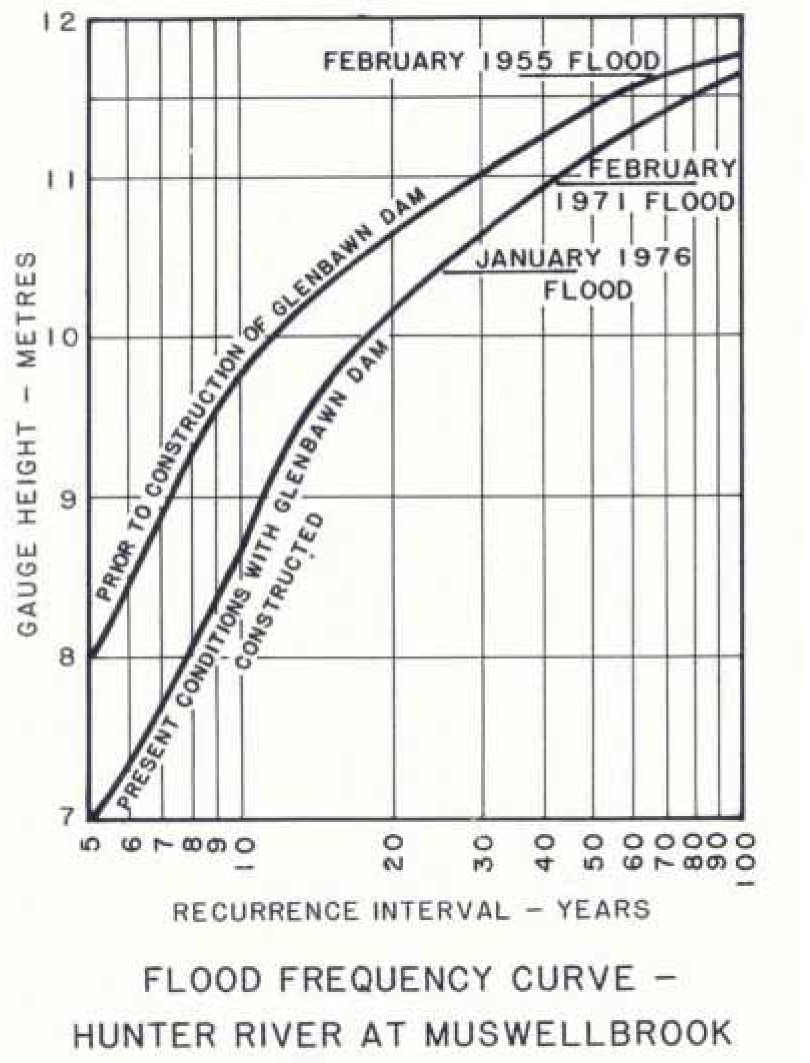


Diagram 1 Copy of Frequency Curve on Muswellbrook Flood Inundation Map

Diagram 1 indicates a reduction of approximately 1 m for all events below 10 m. For larger events the reduction decreases significantly. For a pre-dam gauge height of 11 m the reduction is 0.4 m and for a gauge height of 11.7 m (1955 flood level) the reduction is of the order of 0.2 m. These results are more typical of what might be expected from construction of Glenbawn Dam, rather than those published in Reference 2.

2.2.3. Audit of Flood Levees for New South Wales – Town of Aberdeen, 1994 (Reference 3)

This report undertook an audit of the 700 m long levee which runs on the northern end of the town parallel to the Hunter River. A number of recommendations were made regarding the design of the levee and many of these were addressed in Reference 4.

2.2.4. Aberdeen and Singleton Levees – Phase 1 Study Report, April 1999 (Reference 4)

The report concluded that there were a number of deficiencies with the levee at Aberdeen, including:

- floodwaters can outflank the levee at the downstream end,
- inadequate compaction may increase the likelihood of piping failures,
- the levee is being damaged by stock traffic,
- the trees growing on the levee should be removed.

2.2.5. Scone Flood Study, November 1996 (Reference 5)

This report identifies the nature and extent of flooding along the three main waterways, Middle Brook, Kingdon Ponds and Parsons Gully, located to the west of the Scone town centre. The total catchment area to the limit of the study area is approximately 360 km^2.

A RAFTS hydrologic model was established and calibrated to the Parkville record for the flood events of January 1976, March 1977 and February 1992 with a Bx value (a storage routing parameter used for model calibration) of 0.75. The model assumed a continuing loss of 2.5 mm/hr for all design events except the PMF which assumed 1 mm/hr. Initial losses varied from 10 mm to 80 mm. Design rainfall intensities for Scone were adopted over the entire catchment with no rainfall reduction factor. The data for the February 1992 event was considered to be the most reliable of the three events and a close match was achieved between the predicted and observed peak flows/volumes for this event as indicated in Table 3.

Table 3 Recorded and Modelled Results for February 1992 – Reference 5

	Recorded at Parkville	RAFTS Model
Peak Flow (m^3/s)	452	450
Volume (m^3 x 10^3)	20679	20241
Time to Peak (min)	2070	2190

The critical design storm duration at Scone was determined as 48 hours for the 10% and 5% AEP events and 36 hours for the 2%, 1% and 0.5% AEP events. For the PMF, the 4 hour duration storm generated the largest peak flows. The adopted peak design flows, at the downstream study boundary (Scone), are listed in Table 4.

Table 4 Peak Design Flows at Scone – Reference 5

AEP (%)	Peak Flow (m^3/s)
10	448
5	694
2	958
1	1208
0.5	1430
PMF	8451

Flood frequency analysis of the record at Parkville (or sometimes termed Kingdon Ponds) was also undertaken. This produced a 1% AEP flow approximately 35% greater than the adopted RAFTS flow.

A MIKE11 hydraulic model was established for the floodplain from approximately 5 kilometres upstream of Scone to 6 kilometres downstream. The model was calibrated to historical data (January 1976 and February 1992) and used to determine design flood levels, flows and velocities.

The report indicates that the February 1992 event was of similar magnitude to the February 1955 event.

2.2.6. PMF and Spillway Adequacy Study for Glenbawn Dam, April 1998 (Reference 6)

This report describes an assessment of the adequacy of the Glenbawn Dam spillway, which was made on the basis of revised estimates of the catchment's Probable Maximum Precipitation (PMP). The dam has a catchment area of 1300 km^2 and construction was completed in 1958 but it was subsequently enlarged in 1987. At Full Supply Level (FSL) of 276.25 mAHD the dam has a capacity of 750 gigalitres (GL) (only 300 GL before upgrading). One gigalitre is the volume contained within approximately an area 320m by 320m and 10 metres deep. By comparison Warragamba Dam in western Sydney has a storage capacity of 2000 GL and a much larger catchment area of 9000 km^2. Above the Glenbawn Dam FSL there is an additional capacity of 120 GL of flood storage (this volume could contain approximately 90 mm of runoff over the entire catchment). The uncontrolled spillway with a three bay fuse plug is at 280.6 mAHD.

A hydrological catchment model for Glenbawn Dam was developed utilising the RORB runoff routing model. This model was calibrated against four historical flood events (January 1976, February 1976, March 1977, March 1978), which included the highest event on record (March 1977). The continuing loss model was adopted with initial losses between 0 mm and 32 mm and continuing losses between 1 mm/hr and 4.9 mm/hr. The RORB parameter k_c was set at 14 for the catchment to Moonan Dam and 20.7 for the remaining catchment area. The parameter m was set at 0.8 for the entire catchment. For design the catchment was divided into four different zones to represent rainfall variability over the catchment area. Rainfall reduction factors ranging from 0.6 to 0.9 were applied to the design storm events. The 2% and 1% AEP events were based on ARR87 data (Reference 7). The PMF was determined using both the Generalised Southeast Australia Method (GSAM) and Generalised Tropical Storm Method (GTSM) as the catchment area is located in the 'Transition Zone' between the two methods. The intermediate design rainfall events were evaluated based on an interpolation between the PMF and the 1% AEP event as specified in ARR87.

The study analysed two different initial storage levels in the reservoir. One scenario assumed that the dam is at FSL but the flood mitigation zone is not encroached upon at the time of the commencement of the flood. The other scenario assumed that the PMF was preceded by a large flood and that the water level was at the spillway crest. Flood frequency analysis was also carried out on the Moonan Dam site (735 km^2 catchment area) and Glenbawn Dam flow data.

2.2.7. Glenbawn Dam Assessment on Spillway Adequacy Using a Joint Probability Approach, April 2001 (Reference 8)

This report describes the review of the spillway adequacy of Glenbawn Dam using a joint probability analysis approach and utilised the same RORB hydrological model as in the previous study (Reference 6). However slightly different losses were assumed for events less frequent than the 1% AEP. Whereas the previous study assumed two initial reservoir levels, this study examined a range of initial water levels and the associated probability of each reservoir level.

The two elements of the joint probability analysis were:

- a frequency curve showing the probability of exceedance of initial levels in the dam. This was derived using a monthly simulation of the storage of the dam under current demand conditions,
- inflow hydrographs of various frequencies and durations.

The joint probability approach was used to estimate the outflow frequency curve. The initial reservoir level and the peak inflows were assumed to be independent.

A key feature of the results is that for events greater than a 0.2% AEP event the Joint Probability outflow (e.g. PMF = 12,870 m^3/s) is smaller than the FSL outflow (e.g. PMF = 14,285 m^3/s). Also, the Joint Probability outflow PMF will not exceed the capacity of the dam's spillway. A comparison of the peak design outflows from the two studies (References 6 and 8) is shown in Table 5. The Joint Probability peak outflows are lower than the peak outflows determined in the earlier study. This outcome reflects the fact that the Joint Probability assessment takes into account the possibility that the initial water level at the start of the flood is lower than the FSL.

Table 5 Comparison of Peak Design Outflow from Glenbawn Dam (References 6 and 8)

AEP (%)	Peak Outflow (m^3/s)		
	Initial Storage at FSL – Reference 6	Initial Storage at Spillway Crest - Reference 6	Joint Probability Approach - Reference 8
2	130	1060	0
1	237	1384	0
0.05	2607	4534	1457

There appears to be some inconsistencies between the two reports as the second study reports FSL results somewhat lower than those obtained in the initial study. Also, the peak outflows based on the Joint Probability approach are in some instances greater than the reported FSL outflows. This result seems questionable as the Joint Probability approach assumes that the initial storage is lower than the FSL for 80% of the time.

2.3. Causes of Flooding

Flooding within the Hunter River at Aberdeen may occur as a result of a combination of flows from the following main tributaries:

- Hunter River spilling from Glenbawn Dam (1300 km^2),
- Rouchel Brook (434 km^2),
- Pages River including the Isis River (1177 km^2),
- Kingdon Ponds (363 km^2),
- Dart Brook (426 km^2),
- local catchment inflows not included above.

Flooding in the major tributaries may occur in isolation or in combination with each other. Generally the peak flow in each tributary will not be coincident with each other due to the varying catchment size and certainly in historical events the differing temporal patterns of rainfall over each catchment.

Flooding at Aberdeen may also result from a “sunny day” dambreak at Glenbawn Dam. This mechanism or any type of dambreak (or any dam) has not been analysed as part of this Flood Study.

3. DATA

The first stage in the investigation of flooding matters is to establish the nature, size and frequency of the problem. On a large river system such as the Hunter River there are stream height and historical records dating back to the early 1900's, or in some cases even further. This information together with an examination of rainfall records and local knowledge has been examined to obtain a picture of flooding at Aberdeen.

3.1. Flood Information Sources

A data search was carried out to identify the dates and magnitudes of historical floods. The two prime sources of data were:

- Pinneena river height/flow records. Pinneena is an electronic database of streamflow/height data published by the Department of Natural Resources (formerly the Department of Infrastructure, Planning and Natural Resources) (Figure 2).
- Bureau of Meteorology (BOM) rainfall records (Figure 3).

The two main objectives for obtaining and reviewing this information are:

- *To obtain a historical record of flooding at Aberdeen.* Which flood was the largest, how often have floods occurred, what level have they reached? Unfortunately there is only a limited historical flood level record at Aberdeen (1998 to 2004) which is not as comprehensive as exists at Muswellbrook, Singleton or Maitland. We have therefore considered the flood record at Muswellbrook (12 kms downstream).

- *To obtain events for calibration/verification of the hydrologic/hydraulic models used for design flood estimation.* The two main factors influencing the decision are:
  - the availability of rainfall (particularly pluviometer – continuous record of rainfall rather than the 24h totals from daily read gauges) and streamflow data. Generally there is better quality and quantity of data for the more recent flood events,
  - the magnitude of the flood event. Preferably the larger floods should be used for calibration as these events are closer to the design events adopted for use in development control by Council.

The past studies (References 5 and 6) used floods in January 1976, February 1976, March 1977, March 1978 and February 1992. Review of the data indicates many problems for the events in the 1970's, such as loss of data during the actual flood (gauge damaged or failed) or unavailability of the data (on Pinneena the early flood height records are not stored - only the flows. This presents problems as there is no way of checking how the flows were obtained.).

The other main source of flood height data within the floodplain near Aberdeen were February 1955 flood levels shown on large plans provided by DNR (now Office of Environment and Heritage). The levels were taken from the maps (converted to AHD using an average conversion of +1.6 m) and are shown on Figure 4.

3.2. Flood Record at Muswellbrook

The flood record (in terms of peak flood flows) at Muswellbrook is documented in Reference 2. No record of the historical flood level is provided although this could be "back" calculated using a rating curve. The flood record at Muswellbrook is complicated due to the construction of Glenbawn Dam and assumptions made in Reference 2 on the attenuation provided by the dam (refer Section 2.2.2). A listing of the largest floods and associated peak flows (to the date of the study - 1984) are given in Table 6.

Table 6 Largest Peak Flows at Muswellbrook to 1984 (pre-Glenbawn Dam) – Reference 2

Year of Flood	Peak Flow (m^3/s)
1870	5920
1955	5020
1864	3970
1971	3870
1893	3110
1976	3190

The above data indicates that the January 1971 and January 1976 floods are significantly less than the 1870 and February 1955 events. The construction of Glenbawn Dam has meant that it is difficult to accurately determine the pre-dam flow. Table 6 indicates (based on historical flood levels for post Glenbawn Dam) that the January 1976 event was the largest since 1972 but only slightly greater than February 1992 and November 2000 (based on peak height records at Muswellbrook shown in Appendix B).

In conclusion, based on the Muswellbrook record, the February 1955 flood at Aberdeen was of the order of 1.3 times greater peak flow than the January 1971 event and 1.7 times greater peak flow than the February 1992 and November 2000 events. These figures are based on a number of assumptions and should only be used as a guide to the relative magnitudes of the peak flows.

3.3. Pinneena Records

Table 7 and Figure 2 provides a listing of all stream gauges upstream of Muswellbrook. A large number of the gauges are of little or no value for this study as:

- some are only flood gauges with no data on Pinneena,
- there are no rating curves (used to convert gauge height to streamflow),
- the only data available are streamflow (not gauge heights). This has occurred (we understand) because of database space restrictions in the past,
- construction of Glenbawn Dam has affected the flood record at the Glenbawn Dam gauge,
- if there is a downstream gauge then generally the upstream gauge will not be used in the analysis,
- some gauges do not record water levels and are only for water quality purposes,
- several gauges have little available data or the data is for events prior to the period of interest,

- data for the gauges on the Glenbawn Dam catchment were not used in this study as this information has already been used in Reference 6 for calibration of the RORB model.

In conclusion data from only seven gauges (highlighted in Table 7) are of value for this study and can be used for calibration purposes.

Appendix B provides a time series of gauge heights (from Pinneena) for the seven key gauges indicated in Table 7. The peak five recorded gauge levels at these gauges are provided in Table 8.

Table 7 Stream Gauges Upstream of Muswellbrook available from Pinneena

Gauge Number	River	Station Name	Catchment Area (km^2)	Flow Data in Pinneena	Height Data in Pinneena	Comment
210032	Dart Brook	Dalmore				No data available in Pinneena *
210088	Dart Brook	Aberdeen No. 2	799	1970-2005	1970-2004	Missing data 1983 to 2002
210058	Dart Brook	Aberdeen No. 1	800	1959-1972		Closed in 1972
210124	Dart Brook	Yarrandi Bridge		1994-2005	1994-2004	
21010057	Hunter	Riverside Site 2				No data available in Pinneena *
21010058	Hunter	Riverside Site 2				No data available in Pinneena *
210139	Hunter	u/s Aldridges		2003-2004	2002-2003	Upstream of Glenbawn Dam *
21010059	Hunter	Tyrells Site 2				No data available in Pinneena *
21010060	Hunter	Tyrells Site 1				No data available in Pinneena *
210038	Hunter	The Glen				No data available in Pinneena *
21010055	Hunter	Thompsons Lane Site 2				No data available in Pinneena *
21010056	Hunter	Thompsons Lane Site 1				No data available in Pinneena *
210005	Hunter	Moonan Flat	751	1913-1961		Upstream of Glenbawn Dam
210018	Hunter	Moonan Dam Site	764	1973-2004	1940-2005	Upstream of Glenbawn Dam
210039	Hunter	Belltrees	1180	1999-2004	1999-2003	Upstream of Glenbawn Dam
210015	Hunter	Glenbawn	1925	1940-2005	1968-2004	Upstream of Glenbawn Dam
210056	Hunter	Aberdeen	4000	1959-2005	1998-2004	
210002	Hunter	Muswellbrook Bridge	4220	1907-2005	1972-2004	Downstream of Aberdeen
210008	Hunter	Muswellbrook Weir	4220	1918-1963		Downstream of Aberdeen closed in 1963
210118	Isis	Stuck Me Up Bridge				No data available in Pinneena *
210070	Isis	Lower Timor	320	1963-1983		Inadequate period of record
210057	Isis	Waverly	443	1959-1964		Closed in 1964
210093	Kingdon Ponds	Near Parkville	177	1972-2005	1987-2004	
210033	Kingdon Ponds	Camyr Allyn	293			No data available in Pinneena
210017	Moonan Brook	Moonan Brook	103	1940-2005	1979-2004	Upstream of Glenbawn Dam
210138	Oaky Creek	At weir		2003-2004	2002-2003	Upstream of Glenbawn Dam *
210019	Omadale Brook	Roma	104	1940-1979		Upstream of Glenbawn Dam
21010068	Omadale Brook	Guy Gullen		2002-2004	2002-2003	Upstream of Glenbawn Dam *
21010062	Pages	Vinery				No data available in Pinneena *
21010061	Pages	Allans Bridge				No data available in Pinneena *
210142	Pages	u/s Kewell Creek				No data available in Pinneena *
210119	Pages	Camerons Dam Site No. 1				No data available in Pinneena *
210140	Pages	Glen Vale		2002-2004	2002-2003	Upstream of Glenbawn Dam *
210081	Pages	u/s Hunter River				No data available in Pinneena *
210061	Pages	Blandford	302	1960-2005	1981-2004	Downstream gauge available
210012	Pages	Cronins	1036	1934-1953		Closed in 1953
210030	Pages	Gundy Bridge	1046	1956-1959		Closed in 1959
210052	Pages	Gundy Recorder	1050	1958-2005	1972-2004	
21010064	Rouchel Brook	Site 1				No data available in Pinneena *
21010063	Rouchel Brook	Site 2				No data available in Pinneena *
210029	Rouchel Brook	Upper Rouchel	246	1950-1980		Downstream gauge available and closed in 1980
210014	Rouchel Brook	Rouchel Brook	395	1934-2005	1981-2004	
210025	Stewarts Brook	Windmere	168	1946-1974		Upstream of Glenbawn Dam
210013	Stewarts Brook	Cloverdale	181	1934-2004	2002-2003	Upstream of Glenbawn Dam
	Used for model calibration in present study and height records provided in Appendix B.					
*	Station not shown on Figure 2 as no co-ordinates available.					

Table 8 The Five Greatest Water Levels Recorded at the Key Gauges

Dart Brook at Yarrandi (1994-2004)		Pages River at Gundy (1972-2004)		Kingdon Ponds at Parkville (1987-2004)	
Gauge Height	Date	Gauge Height	Date	Gauge Height	Date
3.1 m	21/11/1994	7.1 m	23/01/1976	5.1 m	09/02/1992
3.9 m	05/05/1998	6.7 m	30/01/1984	3.2 m	25/01/1996
3.5 m	21/07/1998	8.2 m	09/02/1992	4.3 m	27/07/1998
6.0 m	27/07/1998	7.2 m	25/01/1996	3.5 m	08/08/1998
4.0 m	19/11/2000	6.8 m	20/11/2000	4.8 m	19/11/2000
Pages River at Blandford (1983-2004)		**Hunter River at Aberdeen (1998-2004)**		**Hunter River at Muswellbrook (1972-2004)**	
Gauge Height	Date	Gauge Height	Date	Gauge Height	Date
8.0 m	30/01/1984	7.4 m	21/07/1998	10.3 m	24/01/1976
7.7 m	09/02/1992	6.6 m	28/07/1998	10.2 m	10/02/1992
8.3 m	25/01/1996	8.2 m	08/08/1998	8.8 m	22/07/1998
8.6 m	21/07/1998	6.0 m	06/09/1998	9.7 m	09/08/1998
7.5 m	20/11/2000	9.5 m	20/11/2000	10.0 m	21/11/2000
Rouchel Brook at Rouchel (1981-2004)					
Gauge Height	Date				
3.5 m	07/11/1984				
3.3 m	03/08/1990				
3.1 m	14/09/1990				
4.4 m	07/08/1998				
2.8 m	15/07/1999				

Note: The above record was obtained from Pinneena and may be incorrect if for example, the gauge failed and thus the peak water level was not recorded or the gauge was not in existence.

The dates of major flooding since 1980 as indicated by the historical height data at the above gauges are:

- 29th- 30th Jan 1984,
- 8th- 10th Feb 1992,
- 25th - 26th Jan 1996,
- 20th- 21st July 1998,
- 27th- 28th July 1998,
- 7th- 8th Aug 1998,
- 18th- 22nd Nov 2000.

Reference 5 obtained data for the January 1976 and March 1977 events at Parkville. However these data are not available in Pinneena and have therefore not been used in this study. The data for the above seven dates of major flooding are summarised in Table 9 and Diagram 3 and Diagram 3.

Table 9 Peak Gauge Heights and Flows for Major Floods Since 1980

Gauge		Jan 1984	Feb 1992	Jan 1996	21st Jul 1998	28th Jul 1998	Aug 1998	Nov 2000
Kingdon Ponds near Parkville	Height	n/a	5.1 m	3.2 m	3.3 m	4.3 m	3.5 m	4.8 m
	Flow	-	450 m^3/s	110 m^3/s	135 m^3/s	271 m^3/s	152 m^3/s	386 m^3/s
Dart Brook at Yarrandi	Height	n/a	n/a	2.2 m	3.5 m	6.0 m	missing data	4.0 m
	Flow	-	-	66 m^3/s	170 m^3/s	475 m^3/s	-	215 m^3/s
Rouchel Brook at Rouchel Brook	Height	2.5 m	no significant flooding		2.3 m	2.0 m	4.4 m	1.6 m
	Flow	184 m^3/s	no significant flooding		135 m^3/s	100 m^3/s	663 m^3/s	63 m^3/s
Pages River at Blandford	Height	8.0 m	7.7 m	8.3 m	8.6 m	6.1 m	5.0 m	7.5 m
	Flow	950 m^3/s	867 m^3/s	1036 m^3/s	1200 m^3/s	490 m^3/s	327 m^3/s	811 m^3/s
Pages River at Gundy	Height	6.7 m	8.2 m	7.2 m	5.3 m	4.8 m	4.7 m	6.8 m
	Flow	1134 m^3/s	1390 m^3/s	1210 m^3/s	850 m^3/s	731 m^3/s	730 m^3/s	1150 m^3/s
Hunter River at Aberdeen	Height	n/a	n/a	n/a	7.4 m	6.6 m	8.2 m	9.5 m
	Flow	-	-	-	1020 m^3/s	700 m^3/s	1330 m^3/s	2040 m^3/s
Hunter River at Muswellbrook	Height	8.4 m	10.2 m	7.8 m	8.8 m	8.1 m	9.7 m	10.0 m
	Flow	1150 m^3/s	2245 m^3/s	1170 m^3/s	1587 m^3/s	1300 m^3/s	1960 m^3/s	1870 m^3/s

Notes: Data obtained from Pinneena
n/a data not available
The gauge zero at Aberdeen is 158.81 mAHD

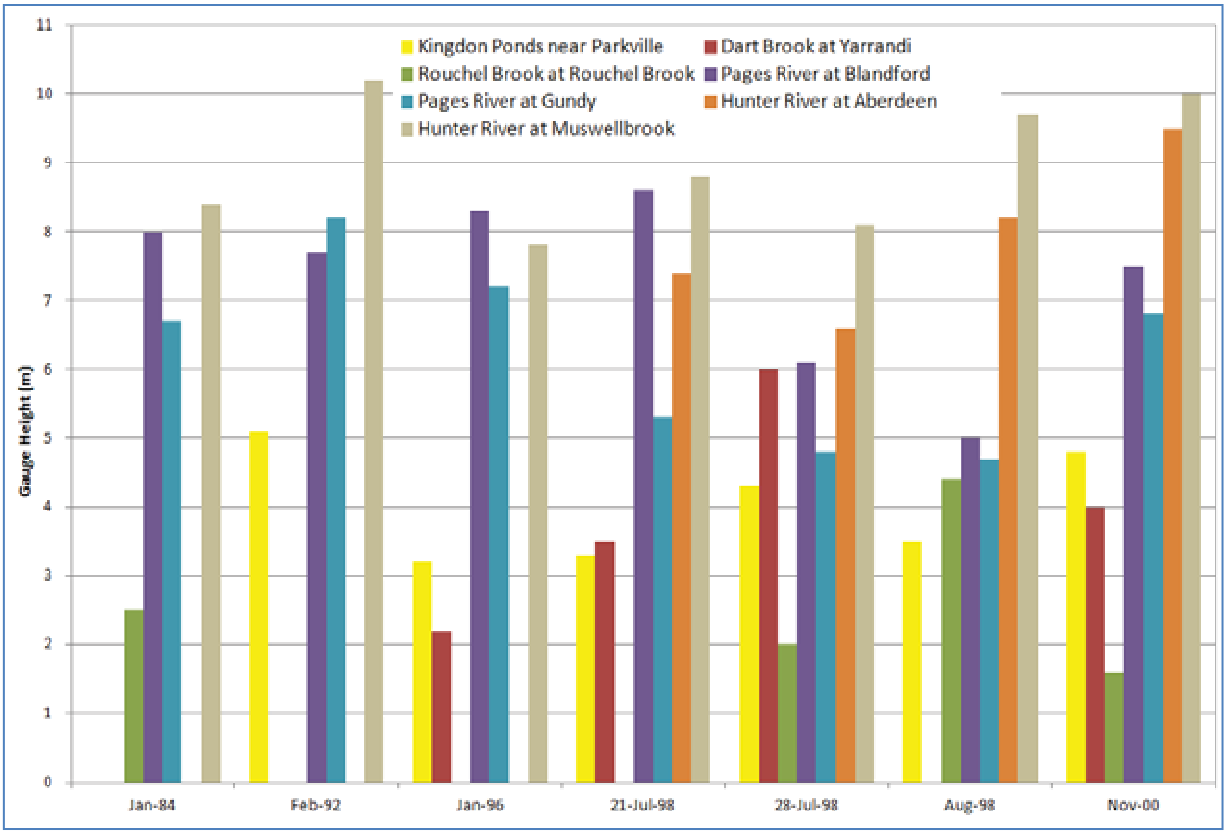


Diagram 2 Graph of Peak Gauge Heights

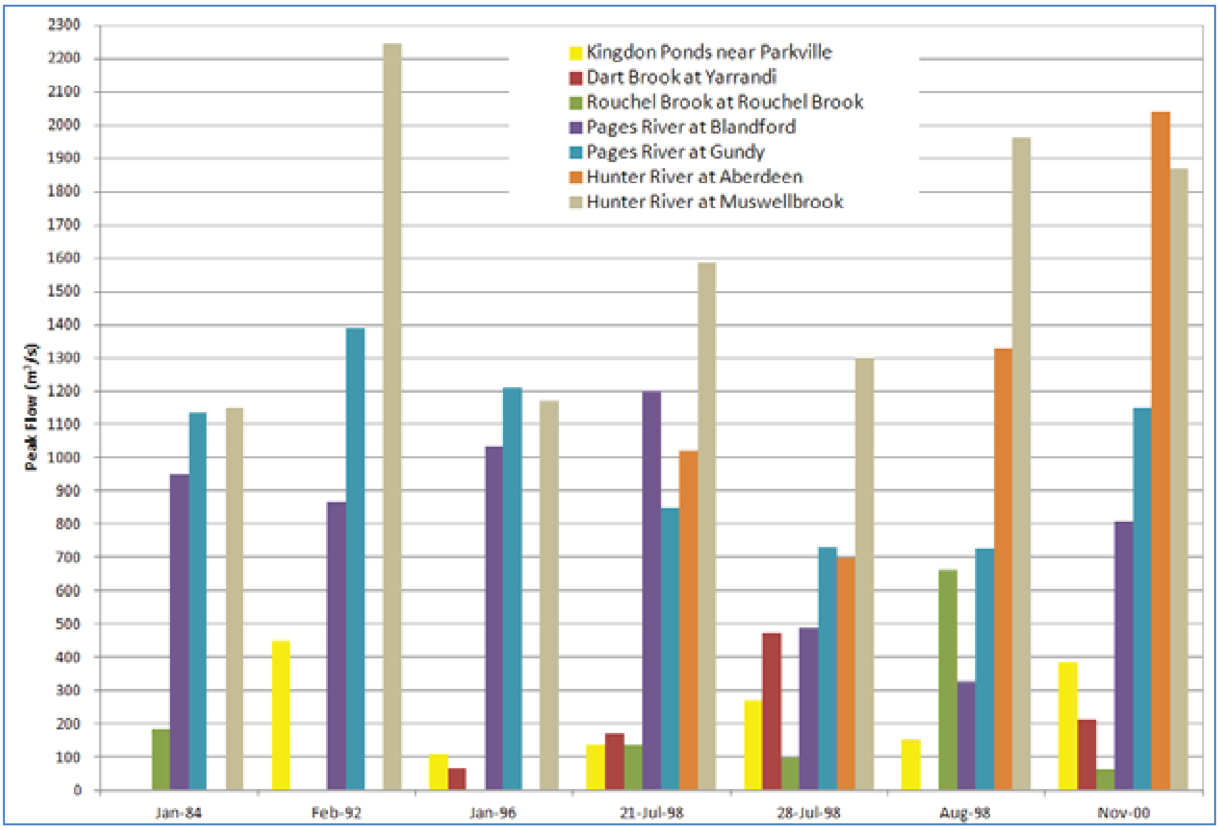


Diagram 3 Graph of Peak Gauge Flows

3.4. Community and Local Resident Survey

In order to identify and collate available flood related information, a survey of all local residents who occupy buildings on the floodplain was conducted as part of the collection of field survey. In terms of past flood events, the survey found that local residents could only identify January 1971 and February 1955 as the major flood events in recent times. There is also additional anecdotal evidence of flooding occurring prior to 1955, however no flood levels are available.

A summary of the information collected is as follows:

- Floodwaters in 1955 reached the window sills of the shops (shown on the front cover of this report) on the New England Highway in the late afternoon. Approximate level = 168.9 mAHD.
- Floodwaters in 1955 almost reached the floor of the Commercial Hotel (the cellar was flooded). Approximate level = 169.7 mAHD.
- Floodwaters in 1955 event reached the 2nd or 3rd step on the house at the corner of Hall and Gundebri Streets. Approximate level = 169 mAHD.
- Floodwaters in 1955 event reached the 3rd step on the café at the corner of McAdam Street and New England Highway. Approximate level = not possible to accurately define.
- The January 1971 flood was more in Dart Brook rather than the Hunter River.
- Floodwaters last entered the residential area of Aberdeen in the January 1971 flood as this was prior to the construction of the levee (constructed in approximately 1976). However there is no information regarding the extent of inundation or damages caused although it is understood that it did not reach the floor of the Golf Club (168.5 mAHD).

3.5. Rainfall

3.5.1. Overview

Rainfall data is recorded either daily (24hr rainfall totals to 9:00am) or continuously (pluviometers measuring depths within small time periods of typically 2 to 5 mins). Together these records provide a picture of when and how often large rainfall events have occurred in the past. Care must be taken however when interpreting historical rainfall measurements. Rainfall records may not provide an accurate representation of past events due to a combination of factors including local site conditions, human error or limitations inherent to the type of recording instrument used. Examples of limitations that may impact the quality of data used for the present study are highlighted in the following:

- Rainfall gauges frequently fail to accurately record the total amount of rainfall. This can occur for a range of reasons including operator error, instrument failure, overtopping and vandalism. In particular, many gauges fail during periods of heavy rainfall and records of large events are often lost or misrepresented.
- Daily read information is usually obtained at 9:00am in the morning. Thus if the storm encompasses this period it becomes “split” between two days of record and a large single day total cannot be identified.
- In the past, rainfall over weekends was often erroneously accumulated and recorded as a combined Monday 9:00am reading.
- Both daily read and pluviometer rainfall records can frequently have “gaps” ranging from a few days to several weeks or even years.
- Pluviometer records provide a much greater insight into the intensity (depth vs time) of rainfall events however there are a smaller number of such gauges. There is only one pluviometer gauge operated by the BOM within the catchment (at Scone Agricultural Station), this has been operational since the mid 1980's. In addition there are four pluviometers operated by OEH at Rouchel Brook, Moonan Dam site, Pages River at Blandford and at Parkville. There is also one BOM pluviometer within close proximity to the catchment at Lostock and OEH pluviometers outside the catchment at Liddell, Barry and Old Warrah. It is unlikely that a single pluviometer will be representative of rainfall patterns occurring over the entire catchment area. Thus it is preferable to have data from more than one pluviometer. Pluviometers can also fail during storm events due to the extreme conditions.

3.5.2. Available Rainfall Data

Table 10 and Figure 3 provide a summary of the official daily read rainfall gauges located close to, or within the catchment. The majority of gauges are operated by the BOM and there may also be other gauges in the area (bowling clubs, schools) but these data have not been collected. In addition data from the seven pluviometers owned by OEH and listed above were obtained from Pinneena. Reference 9 noted that some rainfall data was available from SES rainfall gauges at Murrurundi and Blandford. These data were not collected as part of this study.

Table 10 Rainfall Gauges Located Close to or Within the Catchment

Station No.	Station Name	Year Open	Year Closed	Station No.	Station Name	Year Open	Year Closed
61000	Aberdeen (Main Rd)	1894	open	61202	Waverley	1960	1964
61004	Muswellbrook (Bengalla)	1923	1966	61212	(Power Station)	1964	1992
61007	Bunnan (Milhaven)	1900	open	61212	Liddell (Power Station)	1963	1996
61015	Dangerfield	1933	1965	61229	Piercefield	1903	1912
61016	Denman (Virginia Street)	1883	open	61235	Sandy Hollow (Goulburn Drive)	1964	open
61021	Goorangoola	1885	1967	61241	Carrabolla (Woodberry)	1965	open
61026	Gundy (Miller St)	1887	open	61244	Gundy (Eulalia)	1965	1978
61027	Barsham	1882	1927	61246	Ellerston (Hunters Valley)	1966	1994
61051	Murrurundi Post Office	1870	open	61247	Mt.Royal (Polblue)	1967	1975
61052	Muscle Creek (Clendinning)	1901	1976	61257	Mirannie	1894	1980
61053	Muswellbrook (Lower Hill St)	1870	open	61270	Bowmans Creek (Grenell)	1969	open
61058	Owens Gap (T.O.K.)	1902	1977	61277	Pages Creek	1906	1919
61065	Aberdeen (Rossgole)	1926	open	61278	Moonan Flat (Sempells Rock)	1969	1969
61066	Rouchel Brook	1897	1974	61280	Dunwell	1879	1916
61067	Upper Rouchel School	1926	open	61285	Turanville	1881	1916
61069	Scone (Philip Street)	1873	1992	61286	Redbank 3	1922	1927
61079	Wingen (Murrulla)	1877	open	61288	Lostock Dam **PLUVI**	1969	open
61080	Belltrees 1	1879	1941	61290	Upper Allyn Township	1969	open
61088	Waverley (Belltrees 2)	1887	1926	61292	Eccleston (Masseys Creek (Glengarvan))	1969	open
61089	Scone SCS **PLUVI**	1950	open	61297	Murrurundi (Allston)	1973	1981
61094	Glenbawn Dam	1955	open	61300	Parkville (Aroona)	1971	open
61095	Rouchel Brook (Albano)	1932	open	61305	Muswellbrook (Mirrabooka)	1971	1986
61097	Moonan Flat (High St)	1897	open	61306	Kars Springs (Welldun)	1971	open
61098	Belltrees Homestead	1887	1978	61312	Ellerston (Poitrel)	1970	1972
61099	Blair More 2	1900	1924	61315	Rouchel (Bonnie Doon)	1972	open
61102	Cliffdale	1909	1921	61317	Sandy Hollow (Mt Danger Vineyards)	1972	1975
61104	Ellerston	1884	1944	61320	Tomalla (Ohio)	1972	1973
61105	Glenbie	1898	1942	61321	Gungal (Springfield)	1972	1972
61107	Glen Maynard	1901	1924	61324	Gungal (Merryfields)	1972	open
61109	Kenalea	1894	1928	61325	Upper Allyn (Bald Knob)	1972	1995
61115	St.Clair	1895	1949	61330	Ellerston (Limberlost)	1973	1986
61118	Warkworth 1 Public School	1897	1943	61335	Stewarts Brook Composite	1891	1983
61121	Lostock Post Office	1952	1971	61337	Ellerston (Tubrabucca)	1975	1977
61123	Bundabulla (Muswellbrook)	1915	1915	61342	Bunnan (The Cuan)	1977	open
61124	Dartmouth	1880	1922	61343	Scone SCS 2	1952	1970
61126	Pickering	1919	1920	61346	Hunter Springs (Wondecla)	1971	open
61134	Balaibluan	1961	1968	61348	Gundy (Pages River)	1952	open
61135	Upper Rouchel (Mount View)	1961	open	61356	Ellerston (Hunters Vale)	1894	1986
61144	Carlyle	1960	1969	61360	Scone (Kingdon Ponds)	1987	open
61145	Carrabolla	1960	1964	61363	Scone Airport Aws	1988	open
61146	Carrow Brook	1960	open	61365	Scone (Tantanoola)	1988	open
61148	Central Pages	1960	1970	61372	Blandford (Pages River)	1992	open
61153	Murrurundi (Crawney)	1960	1972	61373	Parkville (Kingdon Ponds)	1992	open
61155	Ellerston 2 Post Office	1960	1972	61374	Muswellbrook (St.Heliers)	1992	open
61157	Kars Springs (Twins Hills)	1960	1971	61392	Murrurundi Gap AWS	2003	open
61163	Hunter Springs	1960	1970	61399	Moonan Brook (Pampas)	2003	open
61166	Lagoon Mountain	1960	1975	210014	Rouchel Brook At Rouchel Brook (The Vale)	1992	open
61168	Muswellbrook (Lindisfarne)	1960	open	210018	Hunter River at Moonan Dam Site **PLUVI**	1991	open
61179	Mullee	1962	1967	210061	Pages River At Blandford (Bickham) **PLUVI**	1985	open
61185	Hunter River (Glenbawn Dam)	1940	open	210076	Liddell **PLUVI**	n/a	open
61187	Rouchel Upper (Mulumla)	1960	1977	210093	Kingdon Ponds Near Parkville **PLUVI**	1991	open
61189	Eccleston (Shellbrook)	1960	1981	210124	Rouchel Brook **PLUVI**	n/a	open
61192	Muswellbrook (Spring Ck) (Castle Vale)	1960	open	208009	Barry **LUVI**	n/a	open
61195	Murrurundi (Timor)	1960	open	419076	Old Warrah **PLUVI**	n/a	open
61196	Ellerston (Poitrel)	1960	open		Yarrandi **PLUVI**	n/a	open

3.5.3. Peak Rainfalls

The 1 day, 2 day and 3 day rainfalls with depths greater than 150 mm, 200 mm and 250 mm respectively for the long term daily read stations in Table 10 were obtained and are summarised in Table 11.

Table 11 Highest One, Two and Three Day Rain Events for the Long Term Daily Stations

One day >150 mm			Two day >200 mm			Three day >250 mm		
Station number	Date	Rain (mm)	Station number	Date	Rain (mm)	Station number	Date	Rain (mm)
61051	16/06/1873	227	61007	15/02/1904	216	61000	26/02/1955	294
61051	17/01/1898	172	61000	01/03/1904	217	61007	26/02/1955	298
61051	14/02/1898	165	61000	25/02/1955	211	61026	26/02/1955	294
61000	29/02/1904	196	61007	25/02/1955	212	61051	26/02/1955	325
61000	24/02/1955	173	61026	25/02/1955	209	61065	26/02/1955	277
61007	24/02/1955	153	61065	25/02/1955	235	61079	26/02/1955	302
61026	24/02/1955	155	61051	24/01/1976	229	61089	26/02/1955	288
61089	24/02/1955	163	61095	20/03/1978	206	61051	24/01/1976	298
61135	07/04/1962	163	61135	20/03/1978	283	61051	25/01/1976	272
61051	14/05/1968	156	61288	20/03/1978	230	61315	19/03/1978	273
61135	25/01/1976	164	61315	19/03/1978	235	61315	20/03/1978	261
61306	25/01/1976	224	61346	20/03/1978	274	61095	20/03/1978	264
61135	20/03/1978	156	61346	21/03/1978	236	61288	20/03/1978	270
61346	20/03/1978	208	61288	04/02/1990	244	61346	20/03/1978	301
61288	07/05/1979	152				61135	21/03/1978	296
61135	11/07/1985	150				61346	21/03/1978	302
61135	10/08/1986	156				61288	04/02/1990	280
61135	14/11/1987	150				61306	09/02/1992	277
61135	04/02/1990	162						
61135	09/08/1998	158						
61135	11/03/2001	150						
61135	09/05/2001	162						

The maximum recorded amount of rain falling within the Upper Hunter catchment in one day was 227 mm on the 16th June 1873, over two days was 283 mm, from the 19th to 20th March 1978 and over three days was 325 mm, from the 24th to 26th February 1955. These rainfall depths are greater than the 1% AEP design depths of 170 mm, 220 mm and 240 mm for events of one, two and three day durations respectively. It is not surprising that the February 1955 event dominated the results in Table 11. The other more recent intense rainfall events in Table 11 were January 1976, March 1978 and February 1990. January 1976 probably did produce significant flooding at Aberdeen, based on the Muswellbrook record. The March 1978 event was concentrated over the south eastern part of the catchment, with much lower rainfall intensities recorded elsewhere in the catchment. As a consequence, the recorded peak gauge level at Muswellbrook was relatively low at 6 m. Surprisingly the February 1992 event was only recorded once (in the 3 day totals) and there is no record for the November 2000 event in Table 11. These two events produced the 2nd and 3rd highest river gauge levels at Muswellbrook since 1972.

3.5.4. Design Data

Design rainfall intensities were obtained from ARR87 (Reference 7) with the PMP derived from both the Generalised Southeast Australian Method (GSAM) and the Revised Generalised

Tropical Storm Method (GTSMR). For design events the rainfall was varied across the catchment. The adopted design rainfall zones are shown on Figure 5 (the sub catchments are shown on Figure 6) and the adopted depths for the 1% AEP event are shown on Table 12 and for all AEPs at Scone (as an example) on Table 13.

Table 12 Design Rainfall Depths for 1% AEP Event

Storm Duration (h)	Merriwa	Wingen	Parkville	Scone	Upper Rouchel	Mid Rouchel	Lower Rouchel	Bottom Rouchel	Scone to Muswell - brook	Aberdeen
12	144	140	136	132	141	139	135	131	128	124
18	167	163	159	154	167	164	159	153	150	145
24	185	182	176	171	189	184	178	170	168	162
30	200	198	191	186	206	201	194	185	183	175
36	213	211	203	198	222	215	208	197	195	187
48	233	232	223	217	246	238	229	216	215	206
72	259	259	249	242	281	270	259	242	242	232

Table 13 Design Rainfall Depths for Scone

Storm Duration (h)	Annual Exceedance Probability (%)									
	1 yr ARI	2 yr ARI	20	10	5	2	1	0.5	0.2	PMF
12	44	57	73	84	98	117	132	148	171	472
18	50	65	85	97	117	136	154	173	200	n/c
24	56	72	94	109	126	151	171	193	223	618
30	60	78	101	118	137	164	186	209	242	n/c
36	63	82	107	125	146	174	198	223	258	713
48	69	90	117	138	160	191	217	244	283	818
72	76	99	130	154	178	213	242	273	317	975

Design rainfall information for flood estimation is generally made available to designers in the form of *point rainfall intensities* (as shown in Table 12). However flood estimates are required for catchments of significant size and thus require a design estimate of the *areal average rainfall intensity* over the catchment. The ratio between the design values of areal average rainfall and point rainfall, computed for the same duration and AEP, is called the *areal reduction factor*. It allows for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms over the whole of the catchment area. The WBNM model has an inbuilt aerial reduction function based on the information provided in Reference 7 and this was adopted for all design events.

3.6. Survey

The following survey was obtained as part of this study:

- *Aerial Photogrammetry and Mapping*. This was obtained from Qasco Surveyors (flown in 2006) and provided as breaklines and data points. This data was processed to produce a grid and cross sections for inclusion in the hydraulic model.
- *Floor Level Survey and Survey of Levee and Railway Line*. This was obtained from Boardman Peasley Pty Ltd in September 2006 to January 2007.
- *Check Survey of Aerial Mapping*. This was undertaken by Boardman Peasley Pty Ltd in

July 2008 and indicated that the aerial photogrammetry from Qasco Surveyors was generally in accordance with the field survey by GPS (refer Appendix D for details).

3.6.1. Aberdeen Levee

The following information was predominantly obtained from References 3 and 4.

The levee was constructed in 1976 by the then Water Resources Commission. It is some 700 m long and extends from the New England Highway bridge to the intersection with the main road east (McAdam Street) to Glenbawn Dam near the old butter factory (refer Figure 9).

Reference 3 indicates a design crest level of the January 1971 flood peak plus 1.0 m, a crest width of 2.4 m, an external batter at 3:1 and an internal batter at 2:1. The levee is of earthen construction (obtained from the adjacent floodplain) and was not compacted to a design specification.

The assumed design grade from upstream to downstream was only 0.4 m and the surveyed crest level in 1992 was above the design level (by up to 0.2 m).

Inspection of the levee in 1994 (Reference 3) indicated the following defects:

- inadequate compaction in sections of the levee,
- the materials used in the construction of the levee are generally considered unsuitable for the construction of permanent water retaining structures,
- levee batters are susceptible to minor slumping,
- trees and shrubs have been planted on the levee, adversely affecting the condition and stability of the levee.

Further survey of the levee crest was undertaken in 2002 and 2006 (as part of the present study) and is shown on Figure 7. There are significant differences between each of the three surveys which cannot be readily explained. Whilst it is generally accepted that the crest will reduce in time due to compaction or erosion it is difficult to understand how the crest can rise (most notably the 2006 survey upstream of the railway bridge). The 2006 survey has been used in the hydraulic model.

The levee only prevents floodwaters entering from the Hunter River and is susceptible to backwater flooding across the New England Highway. There is no ready solution to prevent flooding in this manner apart from raising the Highway.

Based on the available floor level data there are some 94 buildings (42 are denoted as multiple buildings and each individual building was not surveyed) that have floor levels lower than the crest of the levee (taken as 170.6 mAHD) within the "leveed" area.

3.6.2. Railway Line

A detailed survey of the railway line was undertaken and is shown on Figure 8. This indicates that the lowest point on the line is at approximately 170.4 mAHD. Details of the 4 openings are as follows (Table 14).

Table 14 Details of Openings through Railway Line

ID*	Description	No. of Openings	Rail Level (mAHD)	Total Width of Openings (m)	Height (m)	Invert (mAHD)
P01	Northern (1 cell)	1	171.2	4.2	2.7	167.5
P02	Northern (4 cell)	4	170.8	16.8	2.2	167.7
O06	Hunter River	6	173.6	136	10+	162.2
O04	Southern (5 cell)	5	173.6	35.6	5.0	167.6

* *See Figure 9 for locations*

4. APPROACH ADOPTED

Diagrammatic representation of the flood study process is shown in Diagram 4. The WBNM hydrologic model (WBNM - Reference 10) was established for the entire catchment (except for the Glenbawn Dam catchment - Figure 6) and used to convert rainfall data into stream flow for input to a hydraulic model (TUFLOW – Reference 11) of the Hunter River (Figure 9). The extent of the hydraulic model was from approximately 9 kilometres upstream of Aberdeen to 12 kilometres downstream of Aberdeen on the Hunter River. To ensure confidence in the results, both models require calibration and verification against observed historical events. The calibrated TUFLOW model was then used to quantify the design flood behaviour for a range of design storm events up to and including the Probable Maximum Flood (PMF).

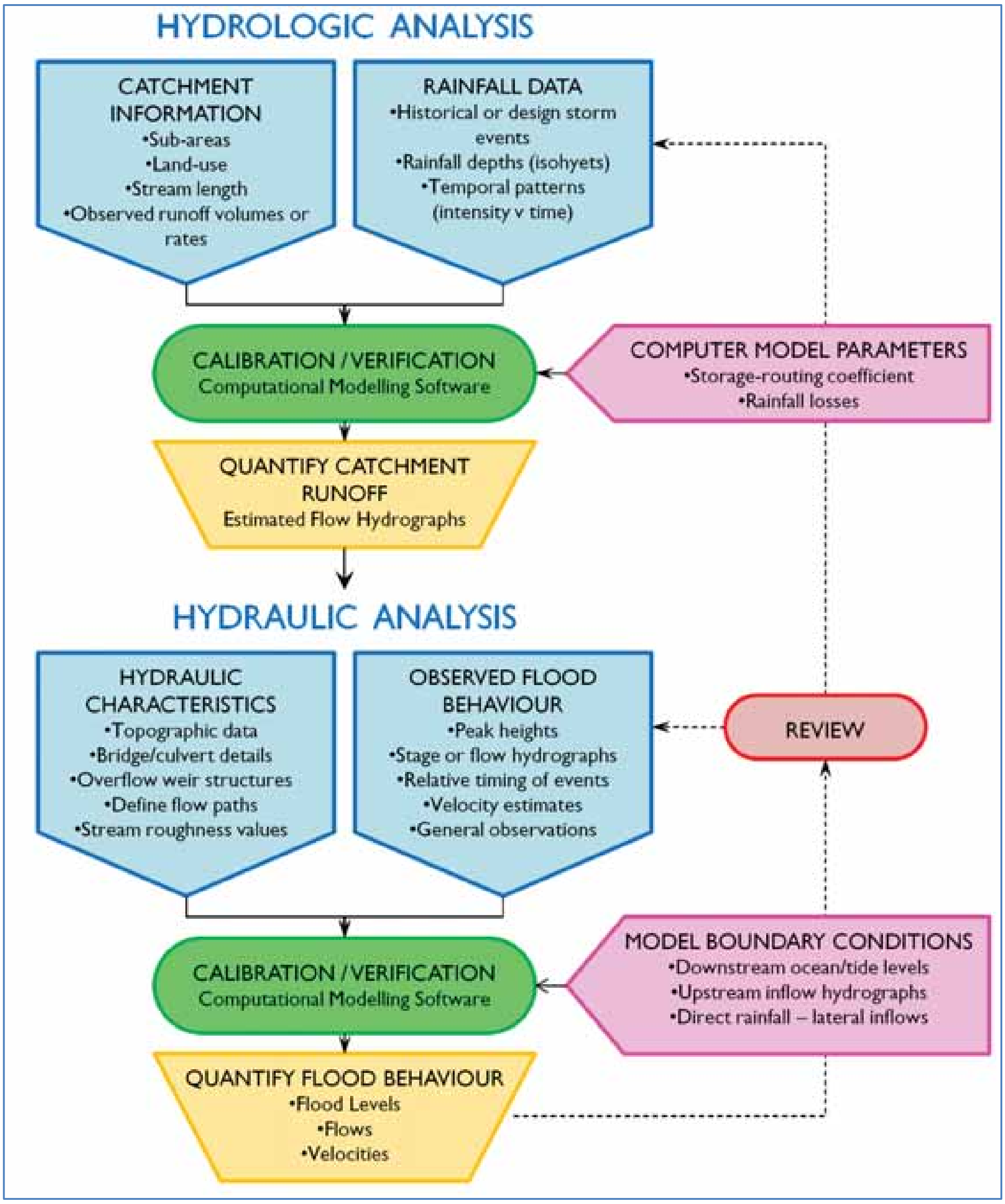


Diagram 4 Flood Study Process

5. HYDROLOGIC MODELLING

5.1. General

Hydrologic models suitable for design flood estimation are described in ARR87 (Reference 7). In current Australian engineering practice, examples of the more commonly used runoff routing models include Watershed Bounded Network Model (WBNM - Reference 10), RORB (Reference 12) and RAFTS (Reference 13). These models allow the rainfall depth to vary both spatially and temporally over the catchment and readily lend themselves to calibration against recorded data.

The Scone catchment (Reference 5) has been previously modelled using RAFTS to provide design flows. For the present study, the WBNM model has been utilised as this would allow a comparison with the previous RAFTS approach. There is no additional advantage in using RORB as the differences between the models are largely eliminated if model calibration to flow data is undertaken.

5.2. Background

5.2.1. WBNM Model

The WBNM model simulates a catchment and its tributaries as a series of sub catchments based on watershed boundaries linked together to replicate the rainfall/runoff process through the natural stream network. The adopted sub catchment division is shown on Figure 6. The key model input data is the area of each sub catchment. For rural catchments no other parameter is required.

The model established for this study comprises a total of 118 sub catchments and included all tributaries upstream of Muswellbrook, excluding the Glenbawn Dam catchment. Outflows from Glenbawn Dam were taken from the RORB model used in Reference 8. The layout of the sub catchments in the WBNM model was defined to provide a reasonable level of spatial detail within the catchment and to provide flow hydrographs at specific locations. For example, the model was structured to provide primary inflows at the upstream limits of the hydraulic model. Catchment areas were determined from topographic contours provided by Council in GIS format. As far as possible each sub catchment was of similar size. No impervious areas were included in the model due to the rural nature of the catchment.

5.2.2. Glenbawn Dam

Flows from Glenbawn Dam were not included in the WBNM model as the effects of the dam have been determined in References 6 and 8. Reference 6 considered that the dam was at FSL before onset of the design flood. Reference 8 assumed a joint probability approach which looked at the coincidence of the dam water level and a range of design durations. The results from Reference 8 (Joint Probability Approach) have been adopted for use in this study as these are the results used for assessment of the dam spillway capacity.

Table 10 of Reference 8 indicates that there is no outflow from the dam for events up to the 0.2% AEP except for outflow from the twin outlet values (each 64 m^3/s). The peak outflow in the PMF is 12,900 m^3/s and this is below the peak capacity of the spillways (14,970 m^3/s). No 0.5% AEP event was simulated in Reference 8 but it is assumed that there is no overflow in this event as the 36 hour duration (critical duration at Aberdeen) indicates no overflow up to a 0.1% AEP event (for the joint probability approach only).

Glenbawn Dam has a significant impact upon flows from the Hunter River catchment at Aberdeen. The brief requested an assessment of outflows for differing initial storage levels in the dam. As this work has been thoroughly investigated in Reference 8 no further work has been undertaken in this study.

For the PMF we have obtained an outflow hydrograph from Glenbawn Dam based on the joint probability analysis given in Reference 8.

5.2.3. Joint Probability Analysis

Given the large catchment area (4,000 km^2) upstream of Aberdeen, floods at Aberdeen can occur as a result of various combinations of inflows from the four major tributaries; Rouchel Brook, Hunter River - Glenbawn Dam, Pages/Isis Rivers, Kingdon Ponds/Dart Brook. The latter joins the Hunter River downstream of Aberdeen and thus does not contribute to flows in the Hunter River at Aberdeen.

The Hunter River catchment also does not contribute due to the construction of Glenbawn Dam (apart from the 128 m^3/s through the twin outlet pipes) up to the 1% AEP event.

Thus the main contributors to the Hunter River flow at Aberdeen are Rouchel Brook and the Pages/Isis Rivers. Joint probability of inflows from these two systems has not been undertaken and it is assumed that the design rainfall would fall across both catchments in the storm event. The areal reduction factor in the hydrologic model accounts for the reduction in design rainfall over a catchment of this size (refer Section 3.5.4).

5.2.4. Approach to Model Calibration

The approach used for calibration of a hydrologic model varies depending on the availability (quality and quantity) of rainfall (daily and pluviometer) and flow data. For Aberdeen there are accuracy issues with both these data sets and particularly with the flow data. This is typical with these types of studies and will only be improved upon as more accurate records of future events become available. For example, generally the rating curve (height v flow relationship) is based on very few flow gaugings, with most at shallow depths. Thus significant extrapolation is required to derive a peak flow for floods which may reach a gauge level several metres above an actual flow gauging. Even a flow estimate (based on velocity measurements) from an actual gauging has probably an error band of the order of ±25%. Thus the error band at higher flows is likely to be even greater.

The key objective of the calibration process is to obtain a WBNM lag parameter and rainfall losses which can be used for design purposes.

Varying the rainfall losses (within reasonable limits) to achieve a calibration to historical data is appropriate as these can vary between storm events. However, varying the lag parameter for different storm events to achieve a calibration is difficult to justify and presents problems when selecting a suitable rainfall losses for design.

Varying the lag parameter and the loss rates across the catchment was considered to achieve a model calibration. Such an approach was rejected as it does not readily produce a simple set of design values to be adopted. Furthermore, it was felt that many of the discrepancies in model calibration were due to the lack of accurate rainfall definition across the catchment and this should not be compensated for by adjusting the model parameters.

Two hydrologic model calibrations have previously been undertaken in the catchment (Scone Flood Study - Reference 5 and Glenbawn Dam Assessment - References 6 and 8). However the latter is not relevant as our present approach is to use the results from the Glenbawn Dam Assessment and not to redo this investigation. The former is relevant and it is appropriate to ensure consistency with the flows in the Scone Flood Study - Reference 5 as they have been used in calibration and verification of a hydraulic model. Any significant changes to the design flows at Scone would then mean revising the design flood levels. This exercise could only be justified if there was robust information which confirms such a change is necessary. Our investigation did not provide such a justification.

In conclusion, the adopted approach has been to ensure that the WBNM model can replicate the design flows from the Scone Flood Study - Reference 5 by using a single lag parameter for all events and uniform rainfall losses across the catchment. This lag parameter value and “average” rainfall losses would then be used for design throughout the Hunter River catchment to Aberdeen.

This study has not researched the accuracy of the rating curves at each of the seven gauging stations and has assumed that the data from Pinneena is accurate. However we note that the Scone Flood Study - Reference 5 revised the rating curve for Gauge No. 210093 - Kingdon Ponds at Parkville (refer Table 15).

Table 15 Comparison of Rating Curve at Kingdon Ponds, Parkville

Gauge Height (m)	Reference 5 (m^3/s)	Pinneena (m^3/s)
0.0	0	0
0.5	0	0
1.0	13	5
1.5	24	14
2.0	39	26
2.5	62	43
3.0	96	67
3.5	147	97
4.0	218	135
4.5	314	185
5.0	439	245
5.5	599	359

This present study has adopted the rating curve provided in the Scone Flood Study - Reference 5 as it is assumed that this is more reliable as it was based on field survey at the time of the Flood Study.

5.2.5. Calibration of WBNM to Scone Flood Study RAFTS Results

In calibrating the WBNM model, two main parameters can be varied to achieve a fit to observed data:

- **Rainfall Losses:** Two parameters, initial loss and continuing loss, modify the amount of rainfall excess to be routed through the model storages.
- **Lag Parameter:** The lag parameter affects the timing of the catchment response to the runoff process and is subject to catchment size, shape and slope.

In order to achieve consistency between the previously published RAFTS results (Reference 5) and the present study, the WBNM lag parameter was varied to replicate the published design peak flows from RAFTS. It was not possible to compare to the historical peak flows as we are unsure what historical rainfall patterns or depths were included in the RAFTS model calibration runs.

The rainfall losses and design rainfall data utilised in the RAFTS model for the Scone Flood Study - Reference 5 were included in the WBNM model. The lag parameter was then adjusted so that the WBNM design peak flows at Scone matched those obtained in the Scone Flood Study, for the range of design flood events.

A WBNM lag parameter of 0.92 was found to produce the best match between the WBNM and RAFTS design peak flows. A comparison of the peak flows for the range of design events is shown on Table 16 and indicates an excellent match across the full range of design events.

Table 16 Comparison of Peak Flows at Scone

Event (AEP) and Duration	RAFTS - Reference 5 (m^3/s)	WBNM - Present Study (m^3/s)
10% - 48 hr	448	455
5% - 48 hr	694	711
2% - 36 hr	958	955
1% - 36 hr	1208	1200
0.5% - 36 hr	1430	1423
PMF- 4 hr	8451	7990

It should be noted that the design rainfall data subsequently used in this present study has been slightly modified from those used in Reference 5 to account for areal variability across the catchment (refer Figure 5). Also a rainfall reduction factor for design events has been included for the present study which was not included in the Scone Flood Study.

5.3. Calibration/Verification

5.3.1. Description of Data

The main factor influencing the events chosen for calibration/verification is the availability of pluviometer data. Several pluviometer records are required to adequately represent the temporal pattern of the entire 4,000 km^2 catchment. There are five pluviometers situated within the catchment upstream at Aberdeen and four immediately outside. The pluviometers that have data for the seven dates of major flooding are listed in Table 17 (with a discussion of the data), with the data provided graphically in Appendix C together with the isohyetal maps.

Table 17 Data Available for Historical Events

Data	Jan 1984	Feb 1992	Jan 1996	Jul 1998 (21st)	Jul 1998 (28th)	Aug 1998	Nov 2000
Pluviometer data available	Liddell Scone	Liddell Moonan Dam Scone	Moonan Dam Blandford Parkville Rouchel Brook Yarrandi Scone	Liddell Moonan Dam Blandford Parkville Rouchel Brook Yarrandi Barry Old Warrah Scone Lostock	Liddell Moonan Dam Blandford Parkville Rouchel Brook Yarrandi Barry Old Warrah Scone Lostock	Liddell Moonan Dam Blandford Parkville Rouchel Brook Yarrandi Barry Old Warrah Scone Lostock	Liddell Moonan Dam Blandford Parkville Rouchel Brook Yarrandi Barry Old Warrah Scone Lostock
Quality of pluviometer data	Scone-missing data prior to flood		Scone - missing data prior to flood	Scone - possibly missing data prior to peak			Scone - missing data
Stream gauges missing data	Dart Brook at Yarrandi, Kingdon Ponds at Parkville, Hunter River at Aberdeen	Dart Brook at Yarrandi Hunter River at Aberdeen	Hunter River at Aberdeen	None	None	Dart Brook at Yarrandi	None
Event used for calibration of hydrologic model	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Event used for calibration of hydraulic model	No	No	No	No - no record at Aberdeen	No - event too small	Yes	Yes

January 1984: The main issue with this event is the lack of pluviometers. Scone recorded

50 mm and Liddell over 75 mm.

February 1992: Liddell, Moonan Dam and Scone all show similar pluviometer patterns and rainfall depths (110 mm, 120 mm and 120 mm). The Moonan Dam record was adjusted to replicate the rainfall totals from the nearby daily read stations as the original record (over 300 mm) was clearly wrong.

January 1996: Four of the six operating pluviometers showed similar patterns with the depths varying from 10 mm to over 65 mm. Scone indicated no significant rainfall burst (total rainfall of 20 mm). Little rain was recorded at Rouchel Brook.

July 21st 1998: All ten pluviometers operated and showed similar patterns. Lostock, Barry and Moonan Dam showed less than 50 mm with the remainder recording between approximately 50 mm and 75 mm.

July 28th 1998: All ten pluviometers operated and showed similar patterns with depths ranging from 25 mm to over 70 mm.

August 1998: All ten pluviometers operated and showed similar patterns with all except Rouchel Brook showing depths ranging from over 30 mm to 80 mm. Rouchel Brook recorded 120 mm (approximately 25 mm greater than the next highest).

November 2000: All except the Scone pluviometer operated successfully but Moonan Dam showed little rain and is not shown on Figure C7. The rainfall depths varied significantly across the catchment. Blandford recorded over 225 mm but the other gauges ranged from 30 mm to 120 mm.

5.3.2. Results

The adopted approach was to use a “C” lag parameter of 0.92 with the default WBNM non-linearity parameter. The initial and continuing losses were then adjusted to achieve an optimal fit across the seven flow gauges (Figure 2 and Table 7) for the seven events.

The results are provided on Figure 10 to Figure 16 and in Table 18. A tabulation of the pluviometer data adopted in each sub-catchment for each design event is shown in Appendix C.

Table 18 WBNM Peak Flow Results (m^3/s)

Gauge	Event	Jan-84	Feb-92	Jan-96	21-Jul-98	28-Jul-98	Aug-98	Nov-00
Parkville	Observed Peak	n/a	450	110	135	271	152	386
	Modelled Peak	260	375	95	155	256	197	424
	% Difference		-17%	-14%	15%	-6%	30%	10%
Yarrandi	Observed Peak	n/a	n/a	66	170	475	n/a	215
	Modelled Peak	285	402	66	177	331	230	287
	% Difference			0%	4%	-30%		33%
Rouchel Brook	Observed Peak	184	n/a	n/a	135	99	665	64
	Modelled Peak	189	270		268	309	774	111
	% Difference	3%			99%	212%	16%	74%
Blandford	Observed Peak	950	867	1036	1200	490	327	811
	Modelled Peak	437	646	823	306	334	239	911
	% Difference	-54%	-26%	-21%	-75%	-32%	-27%	12%
Gundy	Observed Peak	1134	1390	1210	850	731	730	1150
	Modelled Peak	838	1450	1376	725	673	526	1573
	% Difference	-26%	4%	13%	-15%	-8%	-28%	36%
Aberdeen	Observed Peak	n/a	n/a	n/a	1020	700	1330	2040
	Modelled Peak	955	1648	1315	956	829	1150	1563
	% Difference				-7%	18%	-13%	-23%
Muswellbrook	Observed Peak	1150	2245	1170	1587	1300	1960	1870
	Modelled Peak	1289	2269	1431	1298	1221	1726	1601
	% Difference	12%	1%	22%	-18%	-6%	-12%	-15%
Initial Loss (mm)		30	50	10	10	5	5	5
Continuing Loss (mm/h)		1.0	2.5	2.5	1.0	1.5	1.5	2.5
	Figure No.	Figure 10	Figure 11	Figure 12	Figure 13	Figure 14	Figure 15	Figure 16

n/a = data not available

A comment on each calibration follows:

Jan 1984: The lack of pluviometer and flow data means that the outcomes for this event are inconclusive. The recorded rainfalls are relatively low (less than 100 mm in 24 hours). At Rouchel Brook the modelled peak matches the observed but at Blandford and Gundy the observed is greater than the modelled. The peak at Blandford appears high and is only approximately 20% lower then the peak at Muswellbrook. The modelled hydrographs are all a similar shape to the observed.

Feb 1992: Reasonably good matches were obtained at Gundy and Muswellbrook. At Parkville and Blandford the observed was greater than the modelled but both have similar shapes. At Rouchel Brook the observed flow was less than 30 m^3/s and does not accord with the recorded rainfall depths over the catchment (over 120 mm in 72 hours).

Jan 1996: Reasonably good matches were obtained at all locations where observed data were available, except for Rouchel Brook. The latter can be ignored due to the small peak flow (less than 10 m^3/s). At the upstream gauges at Blandford and Parkville the observed is greater than the model whilst the reverse is true for the downstream gauges at Gundy and Muswellbrook. The model cannot replicate the relatively small increase in the observed peak flow from Blandford to Gundy. The modelled versus observed hydrograph shapes are similar.

Jul 21st 1998: Good matches were obtained at Yarrandi and Aberdeen with poorer matches at Parkville and Gundy. At Blandford the “spiky” nature of the hydrograph appears unusual and may indicate a malfunction. Certainly the rainfall depths and temporal patterns do not indicate such a feature. The 60% increase in peak flow from Aberdeen to Muswellbrook cannot be replicated using the recorded rainfall data. The low observed peak at Rouchel Brook also is not reflected in the recorded rainfall data.

Jul 28th 1998: Reasonably good matches to the peak flow were obtained at Parkville and at Muswellbrook. The Yarrandi observed record indicates an unusual shape with a “flat peak”. This may indicate gauge problems. At Rouchel Brook the relatively small peak flow is not reflected by the rainfall data. Also the nearly 100% increase in peak flow from Aberdeen to Muswellbrook cannot be replicated using the observed rainfall data. This could only be explained by a large inflow from the Dart Brook and Kingdon Ponds catchments and a much smaller inflow from Rouchel Brook, as represented by the observed data but not reflected in the model results. The model results could not replicate the shape of the observed hydrograph at the upstream gauges.

Aug 1998: No good matches were obtained for this event. At Parkville and Rouchel Brook the results were high whilst at Blandford, Gundy, Aberdeen and Muswellbrook the model results are lower than observed, despite the use of low initial and continuing losses.

Nov 2000: A reasonable match was obtained at Blandford but the model results were high downstream at Gundy. At all the upstream gauges the modelled peak is higher than the observed but the reverse is true at the downstream Aberdeen and Muswellbrook gauges. No explanation can be provided for the relatively small observed peak flow at Rouchel Brook, given the depth of rainfall. At Parkville the second large peak is not reflected by the pluviometer data which clearly indicates a much larger first peak. Elsewhere (Blandford, Yarrandi, Gundy, Aberdeen, Muswellbrook), the modelled hydrograph shape is similar to the observed.

In summary a calibration of the hydrologic model has been undertaken at seven gauges for seven events. At some gauges the match is good but at other gauges and in other events the match is poor. These results are not unusual for this type of modelling. Closer matches could easily be obtained by varying the storage routing parameter and/or the loss rates over individual catchments. This approach was considered but rejected in favour of the use of a consistent ‘C’ value than ensures compatibility with the design peak flows at Scone (where a previous calibration was undertaken). It is likely that loss rates do vary across the catchment for a given flood event (and probably within an event) however there are no criteria for determining what they are. The calibration of the hydrologic model should be reviewed following each future major event.

5.4. Probable Maximum Precipitation Hydrology

In developing hydrology for the PMF event it is noted that the total catchment area upstream of Aberdeen is approximately 5000 km^2. This large catchment area precludes the use of the Generalised Short Duration Method which is suitable for areas up to 1000 km^2 and uses storm durations up to 6 hrs.

The Aberdeen study area lies in the Coastal Transition Area. Both the Generalised Southeast Australian Method (GSAM) and the Revised Generalised Tropical Storm Method (GTSMR) must therefore be used (References 18 and 19). The method generating the largest PMF is then recommended for adoption. Table 19 shows the critical PMP storm depths calculated.

Table 19 Average PMP Storm Burst Depths (mm)

Total Average Depth (mm)	24hr	36hr	48hr	72hr	96hr	120hr
GSAM	580	670	730	820	890	NA
GTSMR	610	710	800	960	1100	1160

Adopted losses for PMF estimation are listed below and are consistent with recommendations in Reference 7:

Initial Loss	= 0 mm
Continuing Loss	= 1 mm/hr

Estimation of PMF flows is complicated due to the presence of Glenbawn Dam. The approach adopted was to factor the PMF flows from the Glenbawn Dam catchment to equal the peak PMF outflow of 12,900m^3/s as provided in Reference 8. The catchment area draining to Glenbawn Dam is approximately 1295 km^2 and as the PMP depth estimates are a function of catchment area this scaling to 12,900m^3/s is conservative.

6. HYDRAULIC MODELLING

6.1. General Approach

Given the objectives of the study, the available data and in view of the nature of watercourses and potential flow paths within the study area, a two-dimensional (2D) flow representation provides the most efficient and effective assessment of flood behaviour.

The 2D hydraulic model of the floodplain was established using the TUFLOW software package (Reference 11). The TULFOW model is widely used in flood engineering both within Australia and internationally. It is a proven tool for the dynamic modelling of wide floodplains such as at Aberdeen.

The TUFLOW model layout of the Hunter River extends from 9 km upstream of Aberdeen to 12.5 km downstream of Aberdeen (Figure 9) and uses a 10m by 10m grid cell size. The ground level for each grid is based on the aerial photographic mapping as described in Section 3.6. Note that TUFLOW is unique in that elevation data for each cell is stored at the edges (not centre value) so the effective cell size is actually a 5m grid.

Key features of the digital terrain model (DTM) such as roads, railways and levees were represented as break-lines to ensure the correct elevation was utilised. Where structures were more than 2 cells (20m wide) 2D structures were used which more accurately represent the contraction/expansions losses of the structure. Smaller structures require a 1D structure approach.

Cross section data of the lower reaches of the Hunter River provided by the then DLWC indicated that the photogrammetry survey overestimated the channel bed by approximately 1m. Within the bounds of the water breaklines of the photogrammetic survey the DTM was manually lowered by 1m to account for this.

6.2. Model Calibration and Verification

6.2.1. Suitable Historical Events

The hydraulic model calibration and verification is limited due to the sparseness of the historical flood height data, despite a rigorous data collection program.

Whilst there is a reasonable amount of peak height data across the floodplain for the February 1955 event (Figure 4) the only other flood height record is at the Aberdeen river gauge. This provides a very accurate record at the gauge but provides no information about flood levels elsewhere. A very limited amount of data is available for the January 1971 event (Section 3.4).

The lack of overbank flood height data outside Aberdeen is expected as along the Hunter River the relatively confined nature of the floodwaters in events up to the magnitude of a November 2000 event means that property owners will not be significantly affected (apart from agricultural land inundated). It is only in larger floods such as February 1955 or January 1971 that overbank height data will generally be collected. Due to the passage of time much of this data is now not available.

In summary the only flood height data available for calibration are for the Aberdeen gauge for:

- 21st July 1998,
- 28th July 1998,
- August 1998,
- November 2000 events, and
- February 1955 data across the floodplain.

The 21st and 28th July 1998 floods were omitted from the calibration due to their relatively small magnitude at Aberdeen, however they were used for the hydrologic model calibration.

6.2.2. Results

The comparisons of TUFLOW hydraulic model results versus observed data are shown on Figure 17 and Figure 18 for the August 1998 and November 2000 events. It is noted that for the August 1998 event that the water level in Glenbawn Dam was below the active capacity level so no outflow from Glenbawn Dam occurs. For the November 2000 event, the Glenbawn Dam flow is assumed equal to the peak pipe outflow of approximately 128m^3/s.

A comparison of Figure 17 and Figure 18 indicates that in August 1998 the observed and modelled peak flows are similar but the observed and modelled levels differ (by up to 1m). This difference (flow match but not peak height match) is explained by the observed results being estimated using a different rating curve (relationship between height and flow) to that estimated using the TUFLOW model. For the observed results the rating curve is based on an extrapolation of measured flow gaugings (velocities estimated by current meters and waterway area from field survey). In TUFLOW there is no explicit rating curve and the software solves equations that determine flow and velocity for a given water level. For November 2000 the observed/modelled flow match is not as good as for August 1998 but the peak height match is better. This is explained by the observed rating curve having a different "shape" to that estimated using the TUFLOW model.

Records from Pinneena indicate that 250 flow gaugings have been undertaken since 1959. However the largest was for a gauge height of 3.9m (162.7 m AHD) and a flow of approximately 180m^3/s (peak flow in August 1998 was 1300m^3/s). The largest gauging is approximately 4m below the level reached in August 1998 and over 5m below the November 2000 peak level. It is likely that at high flows the TUFLOW model should provide a better estimate of peak flows. The peak flows at Aberdeen could be reduced to provide a closer match, however this would probably mean that the matching of flows at Scone would not agree (Section 5.2.5).

For both events the model water level results are higher than the observed (by 0.4 m in November 2000 and 1.0 m in August 1998). The hydraulic efficiency of the channel and overbank is represented by the stream roughness or friction factor known as Manning's 'n'. This factor describes the net influence of:

- channel roughness,
- channel sinuosity,
- vegetation and other debris in the channel,
- bed forms and shapes.

The above results were obtained with an inner inbank Manning's "n" value of 0.025, outer inbank Manning's "n" of 0.35 and an overbank Manning's "n" value of 0.04. A lower value for the inbank appears inappropriate based on available literature. The model results suggest that the Pinneena rating curve is indicating too high flows at Aberdeen.

It is not possible to use the February 1955 for a model verification event as no pluviometer data are available for this event. However a comparison of the February 1955 recorded flood levels with the 1% AEP peak height profile (Figure 19) was made and indicates a good match through Aberdeen and downstream. Though there are two observed data points upstream (at the Pages River junction and downstream) which appear in error. Upstream of the Pages River junction the match appears reasonable. This comparison suggests that the February 1955 event at Aberdeen was approximately a 1% AEP event and that the modelled profile along the river is in general accordance with the slope of the recorded February 1955 flood levels.

A rating curve was included at the downstream boundary of the TUFLOW model, however as the ground levels at this location are some 10 m lower than at Aberdeen any change to the downstream boundary assumptions will have no impact on flood levels at Aberdeen.

7. DESIGN FLOOD RESULTS

7.1. Overview

There are two basic approaches to determining design flood levels, namely:

- *flood frequency analysis* - based upon a statistical analysis of the flood events, and
- *rainfall/runoff routing* - design rainfalls are processed by a suite of computer models to produce estimates of design flood behaviour.

The *flood frequency* approach requires a reasonably complete homogeneous record of flood levels/flows over a number of decades to give satisfactory results. No such long term records were available at Aberdeen. The record at Muswellbrook could be used, however, this would require investigation into the influence of Glenbawn Dam.

A *rainfall/runoff routing* approach using the WBNM results was adopted for this study to derive design inflow hydrographs. These hydrographs then defined boundary conditions to produce corresponding design flood levels within the study area using the TUFLOW hydraulic model. This approach reflects current engineering practice and is consistent with the quality and quantity of available data.

7.2. Model Parameters

Design temporal patterns derived from ARR87 (Reference 7) are included within WBNM. Rainfall depths based on Figure 5 and Reference 7 (see Table 12) were applied across the catchment. WBNM includes an inbuilt areal rainfall reduction factor which was used for all design events.

For all design events, the hydrological model parameters adopted are:

Lag parameter	=0.92 (this value was derived by matching the flows for the Scone Flood Study - Reference 5, refer Section 5.2.5)
m	=0.8
Initial Loss	=30 mm
Continuing Loss	=2.5 m/h
Areal Reduction Factor	=(as per Reference 7 but approximately 0.9)

The adopted loss rates are within the range of values used for model calibration (Section 5.3.2) and in accordance with the guidelines in Reference 7.

For all design events, the key hydraulic parameter is the Manning's *"n"*, the adopted values are:

Inner Inbank	= 0.025
Outer Inbank	= 0.035
Overbank	= 0.045

7.3. Critical Duration Assessment

In order to determine the duration of the design event that produces the greatest peak levels (termed the critical design duration) the WBNM model was run for a range of durations (ranging from 12hrs to 72hrs) for the 1% AEP event. These results indicated that the 30hr, 36hr and 48hr events produced the greatest peak inflows. These three durations were then input to TUFLOW to determine the critical duration (produces the greatest peak level within the study area). Table 20 indicates the modelled peak levels at the locations marked on Figure 9. The 36 hour design event was adopted as the critical storm duration for all locations within the study area and thus all design events (excluding the PMF) were modelled for the 36 hour duration.

By review of PMP peak flows from the WBNM model, the 24 hour event was the critical event for both the GSAM and GTSMR methods. As shown in Table 20 the GTSMR method generated the highest levels in the hydraulic model and was adopted for the PMF design runs.

Table 20 Peak Flood Levels (m AHD) to Determine Critical Design Duration

ID (refer Figure 9)	1%AEP36hr	1%AEP 30hr	1%AEP 48hr	PMF 24hr GSAM	PMF 24hr GTSMR
L01	175.0	174.9	174.9	180.7	180.9
L02	173.5	173.4	173.3	176.9	177.0
L03	172.4	172.4	172.3	175.8	176.0
L04	171.4	171.4	171.3	175.0	175.1
L05	170.6	170.6	170.6	172.4	172.5
L06	171.0	171.0	170.9	174.3	174.4
L07	171.1	171.0	171.0	174.3	174.4
L08	169.2	169.1	169.1	171.3	171.5
L09	168.9	168.8	168.8	171.1	171.4
L10	166.7	166.7	166.7	170.2	170.6
L11	170.6	170.6	170.6	172.7	173.0
L12	167.1	167.0	167.0	170.9	171.2
L13	165.6	165.5	165.4	170.1	170.4
L14	161.2	161.1	161.1	166.2	166.6

Note: **Red** denotes highest level at each location

7.4. Design Events Results

The results for the design events are provided at the locations shown on Figure 9 and a brief description of these locations are:

- Points with a prefix of L (e.g L01) mark locations where peak water level data in mAHD have been provided,
- Lines drawn parallel to flow direction and labelled with prefix of P (e.g P01) mark locations of key structures modelled as 1D elements. Results at these locations are provided as peak flow in m^3/s.
- Lines drawn orthogonal to the flow direction with a prefix of O (e.g O01) mark locations where peak overland flow is measured as m^3/s. Overland flow paths include bridges

modelled as 2D elements.

Peak height profiles for the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% AEP events and the PMF are provided on Figure 19 and Figure 20. Table 21 and Table 22 shows peak flood levels and peak flows respectively at locations marked on Figure 9. Design flood depths and contours are provided on Figure 21 through to Figure 28.

Table 21 Peak Flood Levels (mAHD)

		AEP							
ID	Ground	20%	10%	5%	2%	1%	0.5%	0.2%	PMF
L01	168.0	172.7	173.3	173.9	174.6	175.0	175.4	176.0	180.9
L02	165.4	171.0	171.7	172.5	173.1	173.5	173.8	174.2	177.0
L03	162.9	170.0	170.8	171.7	172.1	172.4	172.7	172.9	176.0
L04	161.2	169.3	170.1	170.8	171.2	171.4	171.6	171.8	175.1
L05	160.9	168.9	169.5	170.1	170.5	170.6	170.7	170.9	172.5
L06	168.2	NW	NW	169.8	170.8	171.0	171.3	171.6	174.4
L07	168.4	NW	169.0	169.9	170.8	171.1	171.3	171.6	174.4
L08	167.6	NW	NW	168.6	168.9	169.2	169.4	169.6	171.5
L09	159.4	167.5	168.2	168.6	168.7	168.9	169.0	169.1	171.4
L10	158.1	165.5	166.2	166.5	166.6	166.7	166.8	167.0	170.6
L11	167.8	169.9	170.1	170.3	170.5	170.6	170.8	170.9	173.0
L12	164.4	165.8	166.1	166.4	166.8	167.1	167.4	167.7	171.2
L13	159.1	164.1	164.4	164.8	165.2	165.6	165.9	166.3	170.4
L14	149.5	159.7	160.2	160.6	160.9	161.2	161.5	161.8	166.6

NW: Not inundated for this event

Table 22 Peak Flows (m^3/s)

	AEP							
ID	20%	10%	5%	2%	1%	0.5%	0.2%	PMF
P01	0	0	22	38	38	43	47	52
P02	0	18	71	117	122	127	133	158
O01	1551	2032	2715	3507	4058	5116	6317	27771
O02	1540	2019	2683	3450	4036	5099	6299	28812
O03	0	0	0	0	0	0	0	270
O04	0	0	3	28	77	188	269	767
O05	0	0	0	0	0	0	0	210
O06	1538	2008	2529	2907	3114	3401	3687	5258
O07	0	0	0	0	5	53	145	4907
O08	0	0	0	374	690	1280	1994	15307
O09	0	0	0	0	0	10	29	1929
O10	1539	2011	2587	3438	4034	5100	6310	28816
O11	2320	3031	3930	4945	5903	7510	9297	37955
O12	2323	3010	3871	4950	5880	7516	9329	38146

Figure 22 indicates that the majority of the town (except for the golf course) is largely flood free for the 10% AEP event. Properties inside the levee are flood affected by the 5% AEP and larger events. The 2% AEP event sees the levee overtopped upstream and downstream of the railway

(Figure 24) and in the 1% AEP the majority of the low lying urban area of Aberdeen is inundated.

7.5. Sensitivity Analyses

While a decision has been made to adopt the previously stated hydrologic and hydraulic model parameters, sensitivity analysis is conducted for the key parameters. If the modelled results are unduly sensitive to a certain parameter then care must be taken in choosing the adopted parameter. Sensitivity to both hydrological and hydraulic model parameters was undertaken and reported as a change in flood level at the nominated locations. All sensitivity analysis was conducted for the 1% AEP 36 hour design event.

7.5.1. Loss Model

Hydrologic modelling adopted the initial/continuing loss model to determine excess rainfall using the following rainfall parameters:

- Initial loss of 30 mm and continuing loss of 2.5 mm/hr.

Sensitivity analysis was undertaken to determine the result of varied loss parameters as indicated below:

- Initial loss of 35 mm and continuing loss of 3 mm/hr (**PlsLos**),
- Initial loss of 25 mm and continuing loss of 2 mm/hr (**MinLos**).

7.5.2. Pervious Lag Parameter

WBNM relies on empirical relationships which describe catchment response which is principally related to catchment area. In an ungauged fully rural (100% pervious) catchment sub-catchment area, standard rainfall losses and design rainfall data are the only required inputs with generally a "recommended" value of the storage routing parameter "C" adopted. For a gauged calibrated model (as adopted for this study) the "C" parameter is the key calibration parameter along with rainfall losses.

Sensitivity analysis was undertaken to determine the result of a varied storage routing parameter than was determined by model calibration:

- Storage routing parameter adopted by calibration was 0.92.

Revised scenarios with an increased and decreased storage routing parameter were run for:

- Pervious lag parameter 1.10 (**PlsLag**),
- Pervious lag parameter 0.74 (**MinLag**)

7.5.3. Manning's 'n'

The key hydraulic parameter available for calibration was Manning's 'n' and the following three Manning's 'n' adopted:

- Inner inbank 0.025, Outer inbank 0.035 and Overbank 0.045.

Sensitivity analysis was undertaken to determine the results of varied Manning's 'n' values of:

- Inner inbank 0.030, Outer inbank 0.042 and Overbank 0.054 (**PlsMan**),
- Inner inbank 0.020, Outer inbank 0.028 and Overbank 0.036 (**MinMan**).

7.5.4. Results

Sensitivity analysis of hydrologic and hydraulic parameters is reported as a change in peak water level as as shown in Table 23. The typical sensitivity is +/- 0.1m which would be considered minor. Locations 1 and 2 indicate a higher level of sensitivity of +/-0.3m. Near the town of Aberdeen and at the railway bridge the sensitivity range never exceeds +/- 0.2m. This gives confidence in the adopted model parameters.

Table 23 Sensitivity Results 1% AEP Event

		Change in flood level in m					
ID	**Base (mAHD)**	**MinLag**	**PlsLag**	**MinLos**	**PlsLos**	**MinMan**	**PlsMan**
L01	175.0	0.3	-0.3	0.2	-0.2	-0.3	0.3
L02	173.5	0.3	-0.3	0.1	-0.1	-0.3	0.2
L03	172.4	0.2	-0.2	0.1	-0.1	-0.2	0.2
L04	171.4	0.1	-0.2	0.1	-0.1	-0.1	0.1
L05	170.6	0.1	-0.1	0.0	-0.1	-0.1	0.1
L06	171.0	0.2	-0.2	0.1	-0.1	-0.1	0.1
L07	171.1	0.2	-0.2	0.1	-0.1	-0.1	0.1
L08	169.2	0.2	-0.2	0.1	-0.1	-0.2	0.2
L09	168.9	0.1	-0.1	0.0	-0.1	-0.1	0.1
L10	166.7	0.1	-0.1	0.0	0.0	-0.2	0.1
L11	170.6	0.1	-0.1	0.0	0.0	-0.1	0.1
L12	167.1	0.2	-0.2	0.1	-0.1	-0.2	0.2
L13	165.6	0.3	-0.3	0.1	-0.1	-0.2	0.2
L14	161.2	0.2	-0.2	0.1	-0.1	-0.2	0.2

7.6. Flood Hazard and Hydraulic Categorisation

The risk to life and potential damages to buildings during floods varies both in time and place across the floodplain. In order to provide an understanding of the effects of a proposed development on flood behavior and the effects of flooding on development and people the floodplain can be sub-divided into hydraulic (effects of development) and hazard (effects of flooding) categories. This categorization should not be used for the assessment of development proposals on an isolated basis, rather they should be used for assessing the suitability of future types of land use and development in the formulation of a floodplain risk management plan.

7.6.1. Provisional Hazard

Hazard is a measure of the overall harm caused by flooding and should consider a number of factors (depth of flooding, velocity of flood waters, access to escape routes, duration etc.). In the first instance Provisional hazard categories can be defined based on the depth and velocity of floodwaters. Provisional flood hazard categories were defined in this study in accordance with the Floodplain Development Manual - Figure L2 (Reference 14) as indicated below.

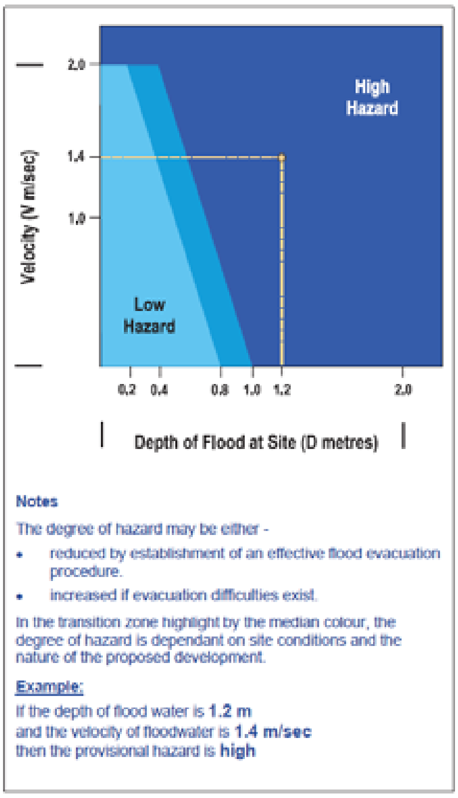


FIGURE L2 - Provisional Hydraulic Hazard Categories

The hazards are provisional because they only consider the hydraulic aspects of flood hazard. Using model results the hazard was calculated from the envelope of the velocity and depth results calculated for each time step. High and low provisional hazard areas were defined for the 5% AEP, 1% AEP and PMF design flood events in Figure 29 to Figure 31.

The Floodplain Development Manual (Reference 14) requires that other factors be considered in determining the "true" hazard such as size of flood, effective warning time, flood readiness, rate of rise of floodwaters, depth and velocity of flood waters, duration of flooding, evacuation problems, effective flood access, type of development within the floodplain, complexity of the stream network and the inter-relationship between flows. The "true" hazard will be determined in the subsequent Floodplain Risk Management Study.

7.6.2. Hydraulic Categorisation

Hydraulic categorisation of the floodplain is used in the development of the Floodplain Risk Management Plan. The Floodplain Development Manual (Reference 14) defines flood prone land to fall into one of the following three hydraulic categories (refer definition in Appendix A taken from Reference 14):-

- Floodway,
- Flood Storage,
- Flood Fringe.

Floodways are areas of the floodplain where a significant discharge of water occurs during floods and by definition if blocked would have a significant affect on flood flows, velocities or depths. Flood storage are areas of importance for the temporary storage of floodwaters and if filled would significantly increase flood levels due to the loss of flood attenuation. The remainder of the floodplain is defined as flood fringe. There is no technical definition of hydraulic categorisation and different approaches are used by different consultants and authorities.

For this study the following criteria have been developed to define the categorisation:

- Flood Fringe:
  Extent for peak depth less than 0.8 m.
- Flood Storage:
  Extent for peak depth greater than 0.8 m.
- Floodway (supersedes Flood Storage when overlapping):
  Extent of peak velocity depth product when greater than 2.0 m^2/s.

Flood Fringe and Flood Storage assume minimal velocity whilst Floodway includes a velocity component. A depth greater than 0.8m is assumed to be high hazard and was adopted to define the difference between Flood Fringe and Flood Storage.

This categorisation may need to be reviewed as there is no absolute definition of Floodway or Flood Storage or Flood Fringe.

Hydraulic categorisation for the 5% AEP, 1% AEP and PMF design flood events is provided in Figure 32 to Figure 34.

7.7. Flood ERP Classification

To assist in the planning and implementation of response strategies, the SES in conjunction with OEH has developed guidelines to classify communities according to the impact that flooding has upon them (Reference 15). Flood affected communities are considered to be those in which the normal functioning of services is altered, either directly or indirectly, because a flood results in the need for external assistance. This impact relates directly to the operational issues of evacuation, resupply and rescue.

Based on the guidelines, communities are classified as either, Flood Islands, Road Access Areas, Overland Access Areas, Trapped Perimeter Areas or Indirectly Affected Areas (refer Table 24). From this classification an indication of the emergency response required can be determined.

Table 24 Emergency Response Classification of Communities

Classification	Response Required		
	Resupply	Rescue/Medivac	Evacuation
High Flood Island	Yes	Possibly	Possibly
Low Flood Island	No	Yes	Yes
Area with Rising Road Access	No	Possibly	Yes
Areas with Overland Escape Routes	No	Possibly	Yes
Low Trapped Perimeter	No	Yes	Yes
High Trapped Perimeter	Yes	Possibly	Possibly
Indirectly Affected Areas	Possibly	Possibly	Possibly

The guideline was applied for the town of Aberdeen. No classification was undertaken for the isolated rural community. The low lying areas of Aberdeen were classified as Low Flood Island based on the following criteria:

- there are homes and access roads below the PMF,
- vehicle evacuation routes are cut before homes are inundated,
- there are no habitable areas for refuge (except the homes themselves),
- the homes are first surrounded by floodwaters and then inundated, and
- thus vehicle evacuation must be completed before the route is closed.

This classification is valid for all events from the 5% AEP through to the PMF.

8. FLOOD RISK, THE SOCIAL IMPACTS OF FLOODING AND CLIMATE CHANGE

8.1. Background

A flood damages assessment was undertaken based upon the floor level survey undertaken as part of this study. The survey included the floor and ground levels for 143 buildings (Table 25 and Figure 35) completed in January 2007. It should be noted that some lots have multiple buildings on them.

Table 25 Floor Levels

Building		Protected by Levee		Floor Level (mAHD)	Number of Buildings
Single Storey Residential	117	Protected by Levee*	115	168-169	66
Double Storey Residential	10	Unprotected by Levee	28	169-170	29
Single Storey Commercial	1			170-171	25
Unspecified Commercial	15			171-172	13
				>172	10
				Total	**143**

* Located south of the Hunter River and east of the Highway

Peak flood levels were determined for each of the buildings for the full range of design flood events. This level was then used with the appropriate formulae and damages curve to determine the tangible property damages for each event.

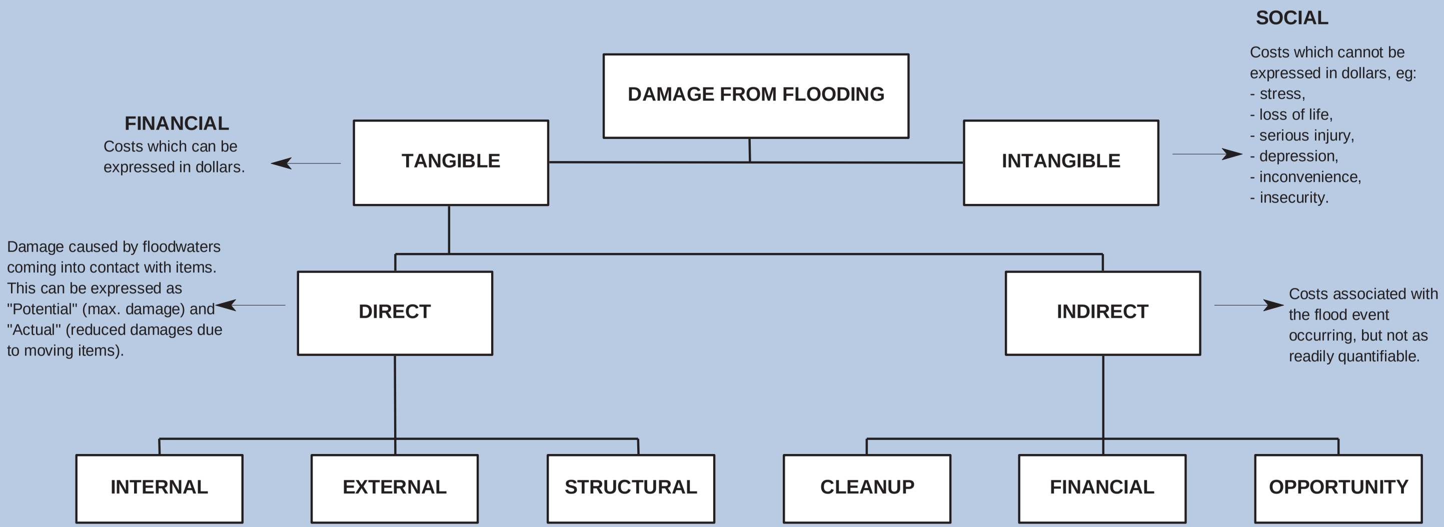
The presence of the Aberdeen levee was also a factor when calculating the flood levels. The levee provides protection from inundation for a number of houses and subsequently influences the damages calculated for each flood event.

The costs of flood damages and the extent of the disruption to the community depends upon many factors including:

- the magnitude (depth, velocity and duration) of the flood,
- land usage and susceptibility to damages,
- awareness of the community to flooding,
- effective warning time,
- the availability of an evacuation plan or damage minimisation program,
- physical factors such as erosion of the river bank, failure of services (sewerage), flood borne debris, sedimentation and wave impacts, and
- the types of asset and infrastructure affected.

Flood damages can be defined as being “tangible” or intangible”. Tangible damages are those for which a monetary value can be assigned, in contrast to intangible damages, which cannot easily be attributed a monetary value. A summary of the types of flood damages is shown on Table 26.

Table 26 Flood Damages Categories



	INTERNAL	EXTERNAL	STRUCTURAL	CLEANUP	FINANCIAL	OPPORTUNITY
RESIDENTIAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Laundries, Caravans, Sheds, Tools, Gardens, Fences	Physical Damage to Buildings: Gyprock, Cupboards, Scour of Footings, Houses becoming buoyant (floating off footings)	Clean Carpets, Walls, Clothes; Re-instate Furniture; Remove Mud and Debris	Loss of wages, Living costs (temporary accomodation and food), Time to repair/replace damaged items	Not Applicable
RURAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Sheds (stables/barns), Machinery, Tools, Fences, Feed storage, Saddles, Crops &/or Stock, Irrigation Systems	Physical Damage to Structures: Damage to Homestead, Sheds, Access tracks, Protection levees	Clean Homestead and Out-buildings; Remove Debris; Dispose of affected crops &/or stock	Loss of Farm Production and Income, Re-instatement of Pastures, Supplementary feeding of stock (by hand or outside agistment), Stock movement/ transport, Living costs (temporary accomodation and food)	Sowing or harvesting of Crops, Sale of Stock (at depreciated value or dependent on market influences)
COMMERCIAL	Contents of Buildings: Products, Stock, Fittings, Tools, Machinery, Raw Materials	External Items: Vehicles, Machinery, Display, Raw Materials/Stockpiles, Fences	Physical Damage to Buildings	Dispose of damaged products, stock, materials; Cleaning and Re-instatement	Loss of Productivity and Income, Bank Interest Charges	Loss of existing &/or Potential Trade
PUBLIC AUTHORITIES	Contents of Public Buildings and Facilities	Public Property and Facilities: Parks, Signs, Machinery, Equipment	Physical Damage to Infrastructure: Electricity, Water, Telephone, Gas, Road & Rail Transport Links	Remove Mud & Debris from Facilities, Public & Private Property Repairs (temporary & permanent)	Disruption of Services, Community Service Relief Grants	Provision of Public Service

8.2. Tangible Flood Damages

Tangible flood damages are comprised of two basic categories, direct and indirect damages. Direct damages are caused by floodwaters wetting goods and possessions thereby damaging them and resulting in either costs to replace or repair or a reduction in their value. Direct damages are further classified as either internal (damage to the contents of a building including carpets, furniture), structural (referring to the structural fabric of a building such as foundations, walls, floors, windows) or external (damage to all items outside the building such as cars, garages). Indirect damages are the additional financial losses caused by the flood including the cost of temporary accommodation, loss of wages by employees etc.

While the total likely damages in a given flood are useful to get a “feel” for the magnitude of the flood problem, it is of little value for absolute economic evaluation. When considering the economic effectiveness of a proposed mitigation option, the key question is what are the total damages prevented over the life of the option? This is a function not only of the high damages which occur in large floods but also of the lesser but more frequent damages which occur in small floods.

The standard way of expressing flood damages is in terms of average annual damages (AAD). AAD represents the equivalent average damages that would be experienced by the community on an annual basis, by taking into account the probability of a flood occurrence. By this means the smaller floods, which occur more frequently, are given a greater weighting than the rare catastrophic floods.

Quantification of tangible flood damages is generally based upon data derived from post-flood damage surveys obtained following historical flood events. An alternative procedure is to undertake a self-assessment survey of the flood liable properties. This latter approach is more expensive and may not accurately reflect what actually occurs in a flood. Floods by their nature are unpredictable and conditions variable. It is therefore unlikely that a self-assessment survey would have predicted the scale or extent of the damages which occurred in Nyngan in 1990 or North Wollongong in August 1998. For this reason it was decided to use the post-flood damage approach in assessing flood damages for the Aberdeen study area.

The most comprehensive damage surveys are those carried out for Sydney (Georges River - 1986), Nyngan (1990) and Inverell (1991). Some of the problems in applying data from these studies to other areas can be summarised as follows

- varying building construction methods, e.g. slab on ground, pier, brick, timber,
- different average age of the buildings in the area,
- the quality of buildings may differ greatly,
- inflation must be taken into account,
- different fixtures within buildings, e.g. air-conditioning units, machinery, etc.,
- change in internal fit out of buildings over the years or in different areas, e.g. more carpets and less linoleum or change in kitchen/bathroom cupboard material,
- external (yard) damages can vary greatly. For example in some areas vehicles can be

readily moved whilst in other areas it is not possible,

- different approaches in assessing flood damages. Are the damages assessed on a "replacement" or a "repair and reinstate where possible" basis? Some surveys include structural damage within internal damage whilst others do not,
- varying warning times between communities means that the potential versus actual damage ratio may change significantly,
- variations in flood awareness of the community.

8.2.1. Tangible Damages – Residential Properties

Tangible direct damages are generally calculated under the following components

- Internal,
- Structural,
- External.

Tangible indirect damages can be subdivided into the following groups:

- accommodation and living expenses,
- loss of income,
- clean up activities.

Damages may be calculated as either estimated actual damages or estimated potential damages. If potential damages are calculated an Actual/Potential (A/P) ratio is assigned based upon (as well as other factors) the likely flood awareness of the community and the available warning time.

The flood awareness of the Aberdeen community is likely to be low given the time since the last flood and because possibly residents now consider themselves "safe" due to construction of the levee. The available flood warning is likely to be at least 12 hours. There is also relatively easy access to high ground for all residents. Based upon the limited data available it is considered that the A/P ratio for the Hunter River would mostly be similar to that applicable at Nyngan and Inverell. However it should be noted that it is impossible to predict the circumstances that may arise during a flood. Also no account was taken of possible failure of Glenbawn Dam.

The approach adopted for estimating flood damages was based on Reference 16 with the results shown on Table 27.

The number of buildings inundated above floor level along with the estimated flood damages are summarised for the range of design flood events in Table 27. Figure 36 shows the distribution of surveyed buildings across the floodplain and also indicates the event first resulting in above floor level inundation.

Table 27 Flood Damages (based on 0 m free board for property flood levels)

	20%	10%	5%	2%	1%	0.5%	0.2%	PMF
Building Floors Inundated	0	0	20	36	85	96	110	130

The Average Annual Damages (AAD) based on the above values is estimated to be $216,000.

8.2.2. Tangible Damages - Non-Residential Properties

Damages to commercial, industrial or public properties cannot be estimated as accurately as damages to residential properties for a number of reasons, including:

- less post-flood surveys have been undertaken in Australia,
- some properties are insured against flood loss, if this is the case the insurance premiums need to be considered in assessing flood damages,
- flood damages can vary greatly from building to building. For example an electrical retail shop may suffer more damages than say a sandwich shop, as the latter has less high value stock. On the other hand there is more opportunity to reduce this actual damage in the former as the items can be easily moved by staff if there is sufficient warning and awareness. In large premises the flood damages depends on the care taken in moving stock. Carpets are high value items and cannot be easily moved whilst the cars in a car showroom can generally be easily moved if there is warning,
- the damages can vary from year to year as the usage of a particular premises changes. Damages may also vary on a seasonal or weekly basis depending upon the nature of business,
- indirect damages (loss of trade) may be significant and these are difficult to properly quantify.

For this study due to the relatively small number of non-residential buildings in the study area and the absence of detailed information about the use of these properties, damages for these buildings were estimated using the same approach for residential buildings.

8.3. Intangible Flood Damages

The intangible damages associated with flooding are inherently more difficult to estimate. In addition to the direct and indirect damages discussed above, additional costs/damages are incurred by residents affected by flooding, such as stress, risk/loss to life, injury etc. It is not possible to put a monetary value on the intangible damages as they are likely to vary dramatically between each flood (from a negligible amount to several hundred times greater than the tangible damages) and depend on a range of factors including the size of flood, the individuals affected, community preparedness, etc. However, it is important that the consideration of intangible damages is included when considering the impacts of flooding on a community. An overview of the types of intangible damages likely to occur at Aberdeen is discussed below.

Isolation

Isolation (the ability to freely exit and enter your house) during flood events will become a significant factor for local residents in outlying areas but within the town is unlikely to be a significant factor. There is also likely to be a high level of community support and spirit, which can to some extent negate the effects of isolation and can certainly assist in a flood. However, isolation is of significant concern if a medical emergency arises during a flood.

Population Demographics

Age, income and unemployment statistics might indicate the possibility of lower resilience of the community to adequately respond to a flood emergency. Well-developed emergency preparedness, response and recovery programs are thus required.

Stress

In addition to the stress caused during an event (from concern over property damage, risk to life for the individuals or their family, clean up etc.,) many residents who have experienced a major flood are fearful of the occurrence of another flood event and its associated damage. The extent of the stress depends on the individual and time or memory of the last major flood (February 1955).

Risk to Life and Injury

During any flood event there is the potential for injury as well as loss of life. Due to the fact that the floodwaters will be travelling across the levee and through urban areas there may be relatively high velocities (even if only as flood waters squeeze between buildings). This velocity component significantly increases the risk to life of people or cars being swept away.

8.4. Climate Change

Climate change is predicted to cause an increase in sea level and possibly changes to design rainfall intensities. The likely impacts of a rise in sea-level can be ignored at Aberdeen.

In developed areas such as Aberdeen, changes in the climate, such as an increase in storm activity are likely to influence future building design, standards and performance as well as planning.

The 2005 Floodplain Development Manual (Reference 14) requires that Flood Studies and Risk Management Studies consider the impacts of climate change on flood behaviour. At the date of this study there is no definitive advice regarding the implications of climate change on design rainfall intensities. Some advice suggests that rainfalls may decrease. In accordance with Reference 17 design rainfall increases of 10%, 20% and 30% were applied to the 1% AEP event and the results are shown on Table 28.

Table 28 Climate Change Rainfall Increase Results 1% AEP Event

		Increase in flood level in m		
ID	Base (mAHD)	CC +10%	CC +20%	CC +30%
L01	175.0	0.3	0.6	0.9
L02	173.5	0.3	0.5	0.7
L03	172.4	0.2	0.3	0.5
L04	171.4	0.1	0.3	0.4
L05	170.6	0.1	0.2	0.2
L06	171.0	0.2	0.4	0.5
L07	171.1	0.2	0.4	0.5
L08	169.2	0.2	0.3	0.4
L09	168.9	0.1	0.2	0.2
L10	166.7	0.1	0.2	0.3
L11	170.6	0.1	0.2	0.3
L12	167.1	0.2	0.4	0.6
L13	165.6	0.3	0.5	0.7
L14	161.2	0.2	0.4	0.6

9. ACKNOWLEDGMENTS

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- Upper Hunter Shire Council,
- Office of Environment and Heritage,
- Floodplain Management Committee,
- Residents of the town of Aberdeen and the Hunter River catchment.

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Figures

FIGURE 1
CATCHMENT MAP

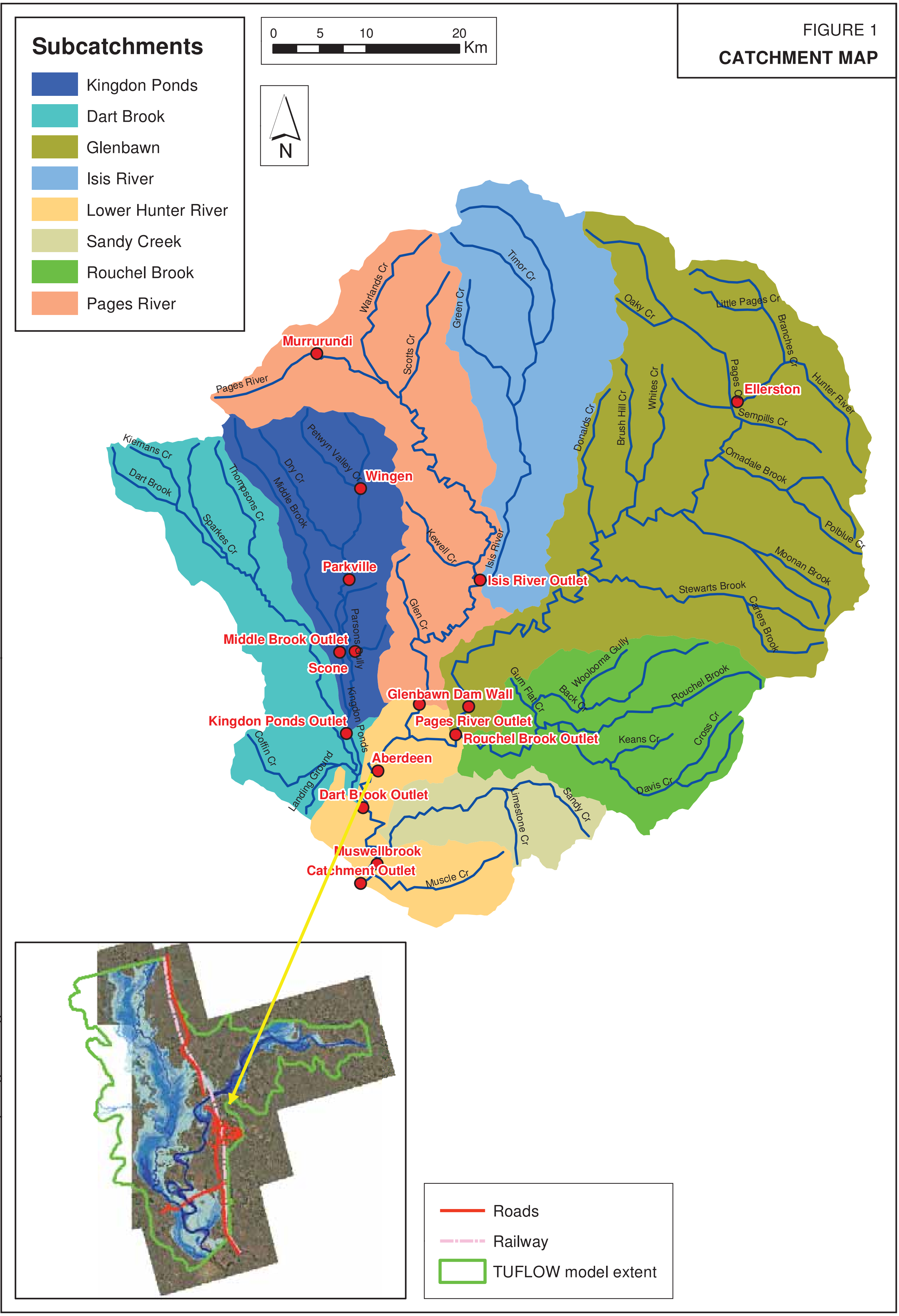

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FIGURE 2

STREAM GAUGING STATIONS

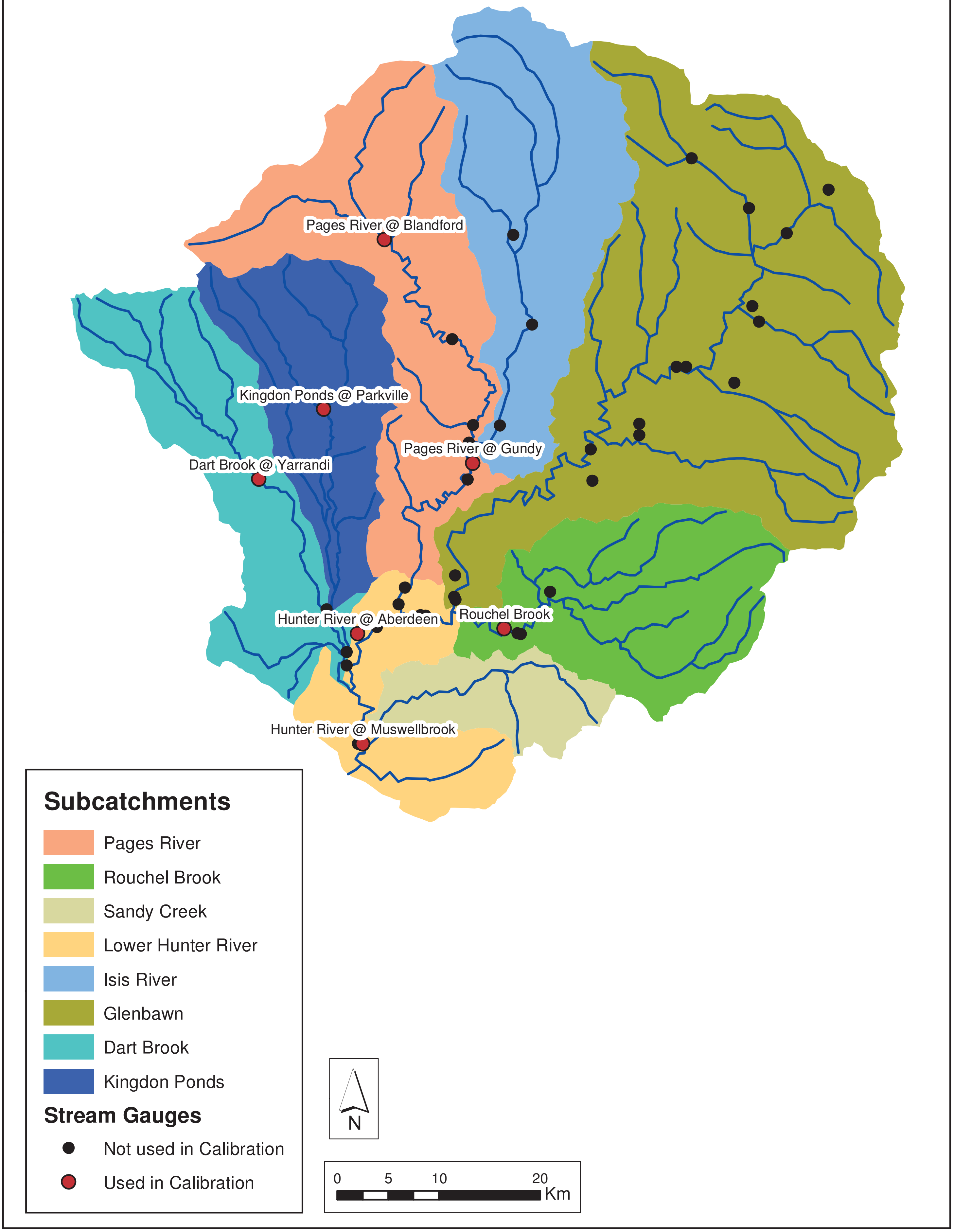
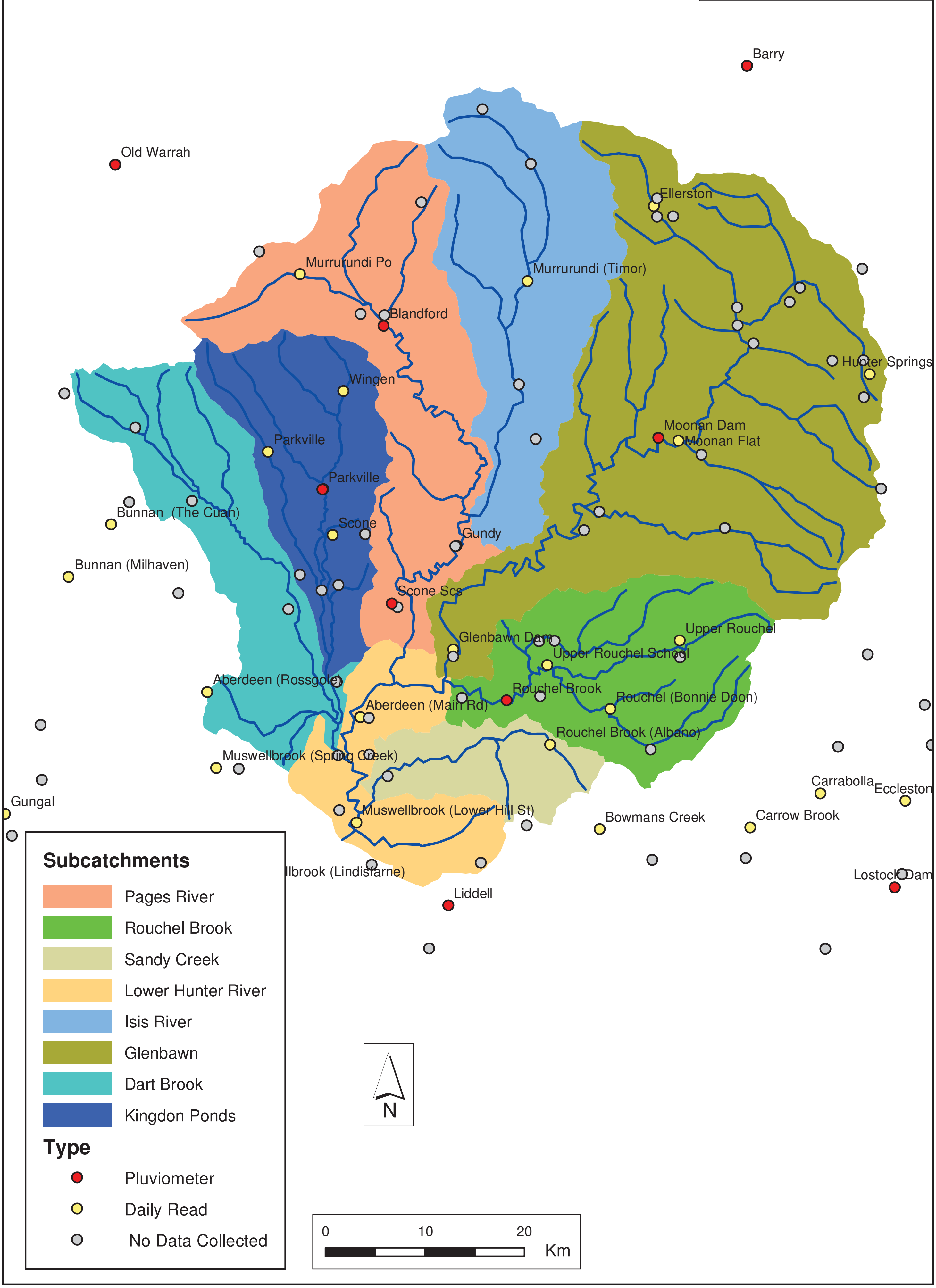


FIGURE 3

RAINFALL GAUGES



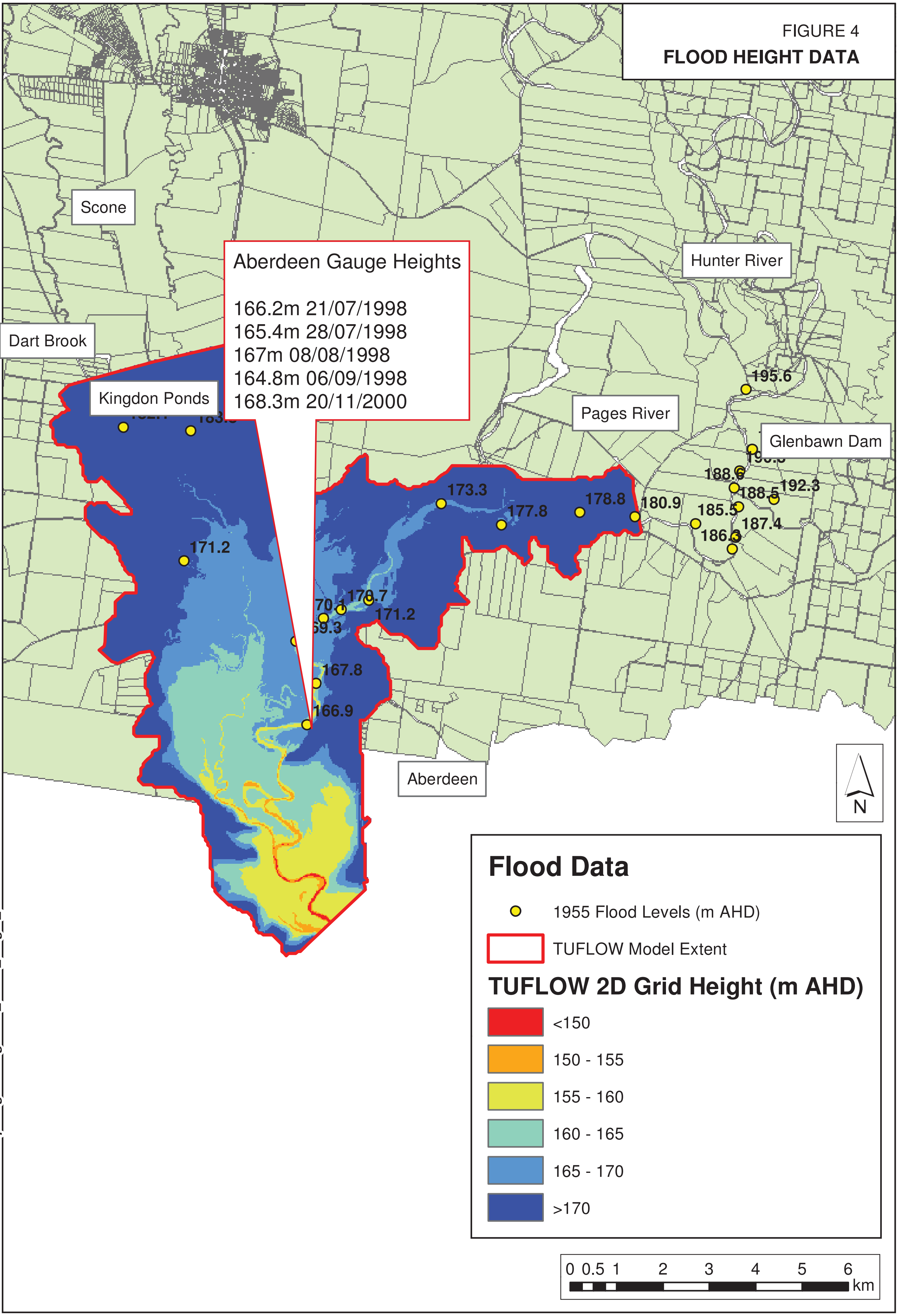


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FIGURE 5

DESIGN RAINFALL ZONES

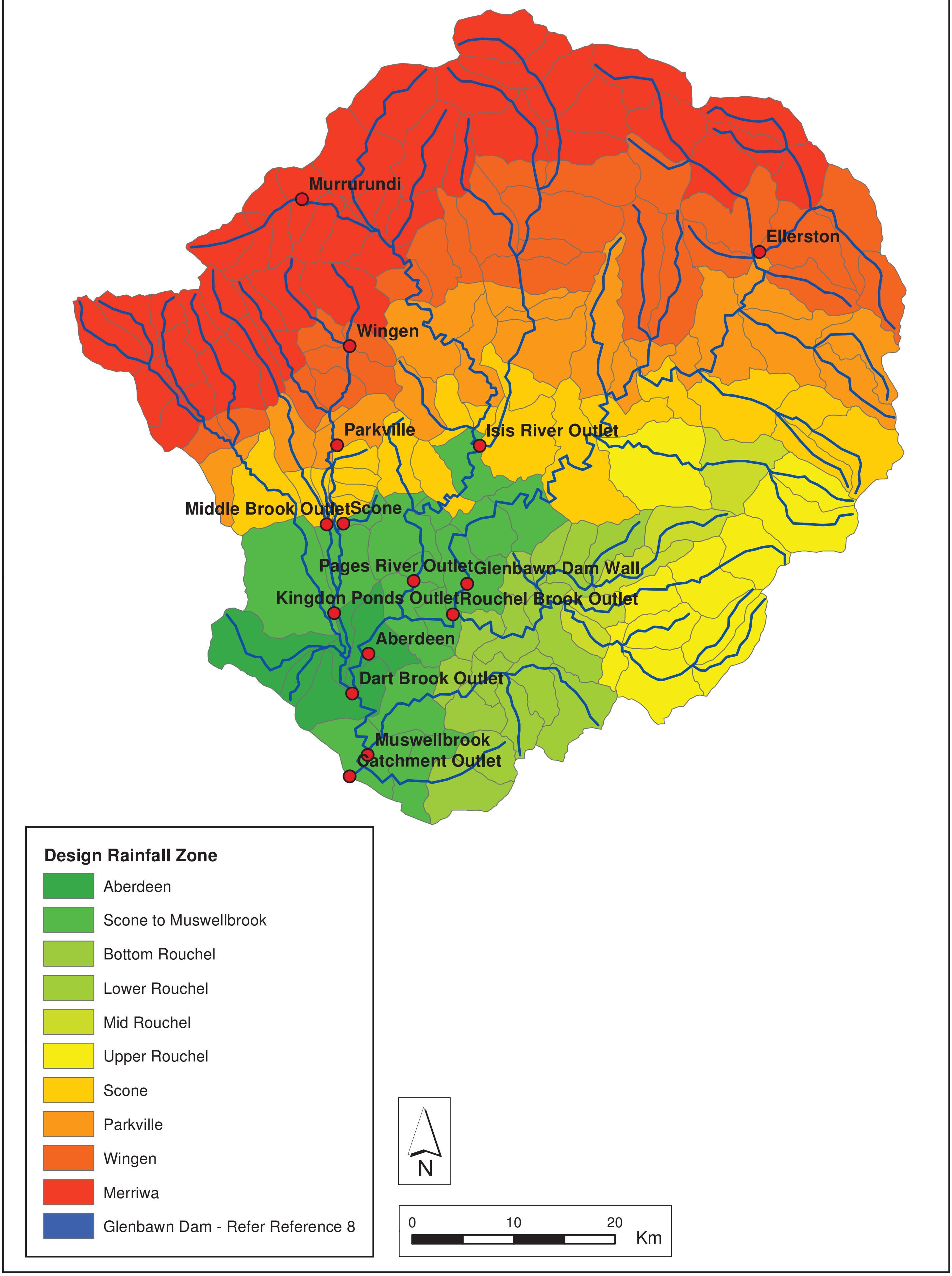

FIGURE 6

WBNM LAYOUT

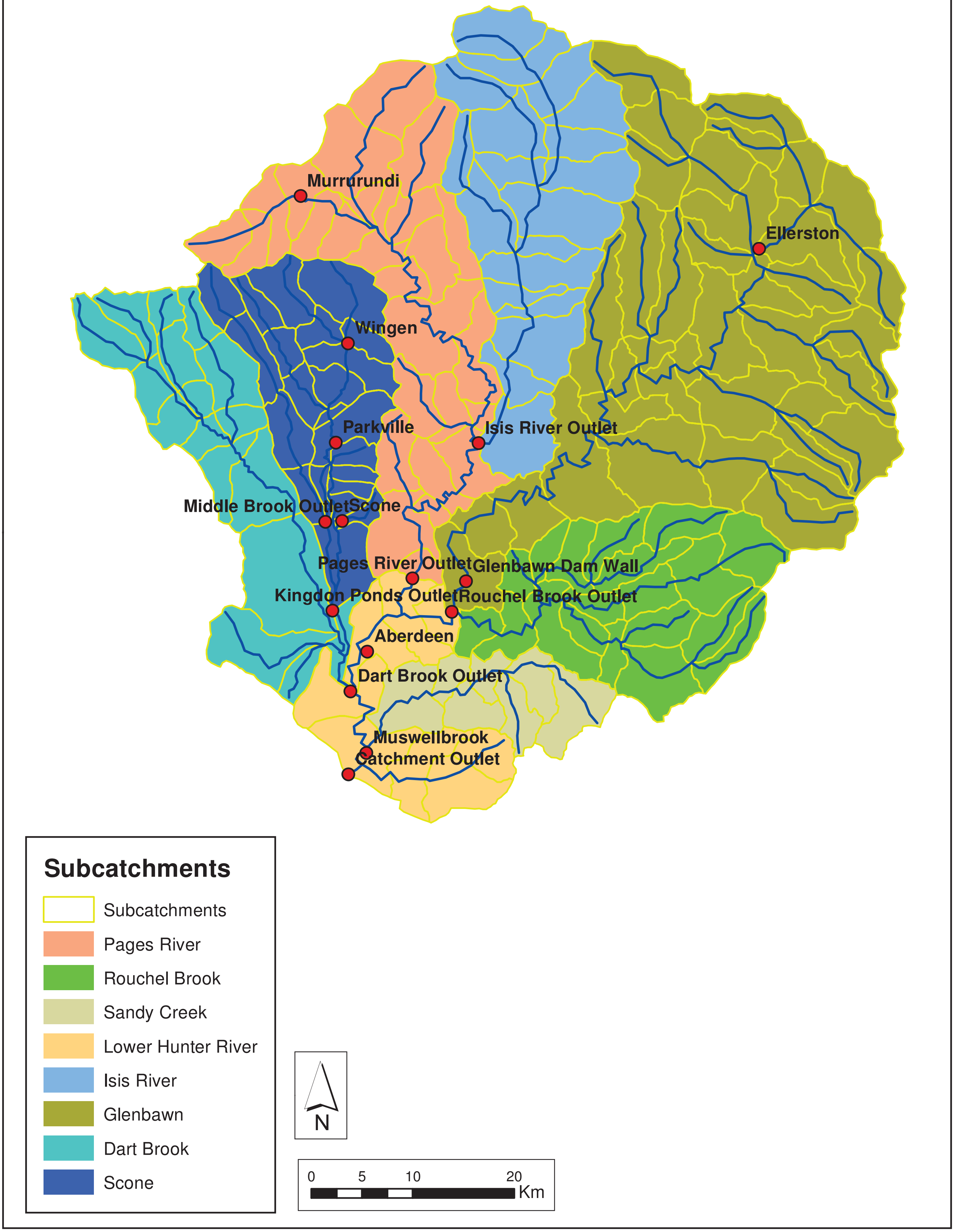

FIGURE 7
SURVEY OF LEVEE CREST

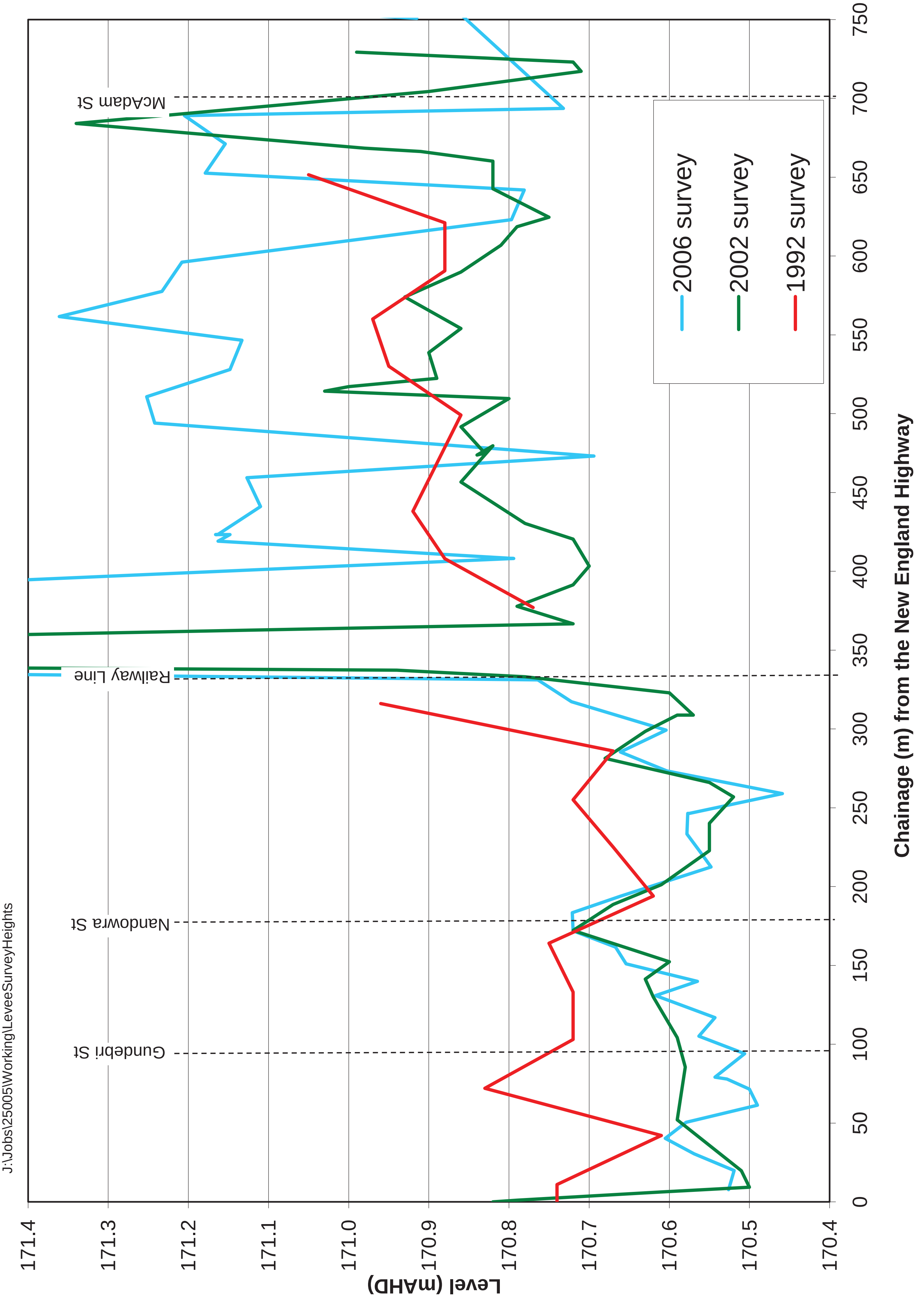

FIGURE 8

SURVEY OF RAILWAY LINE

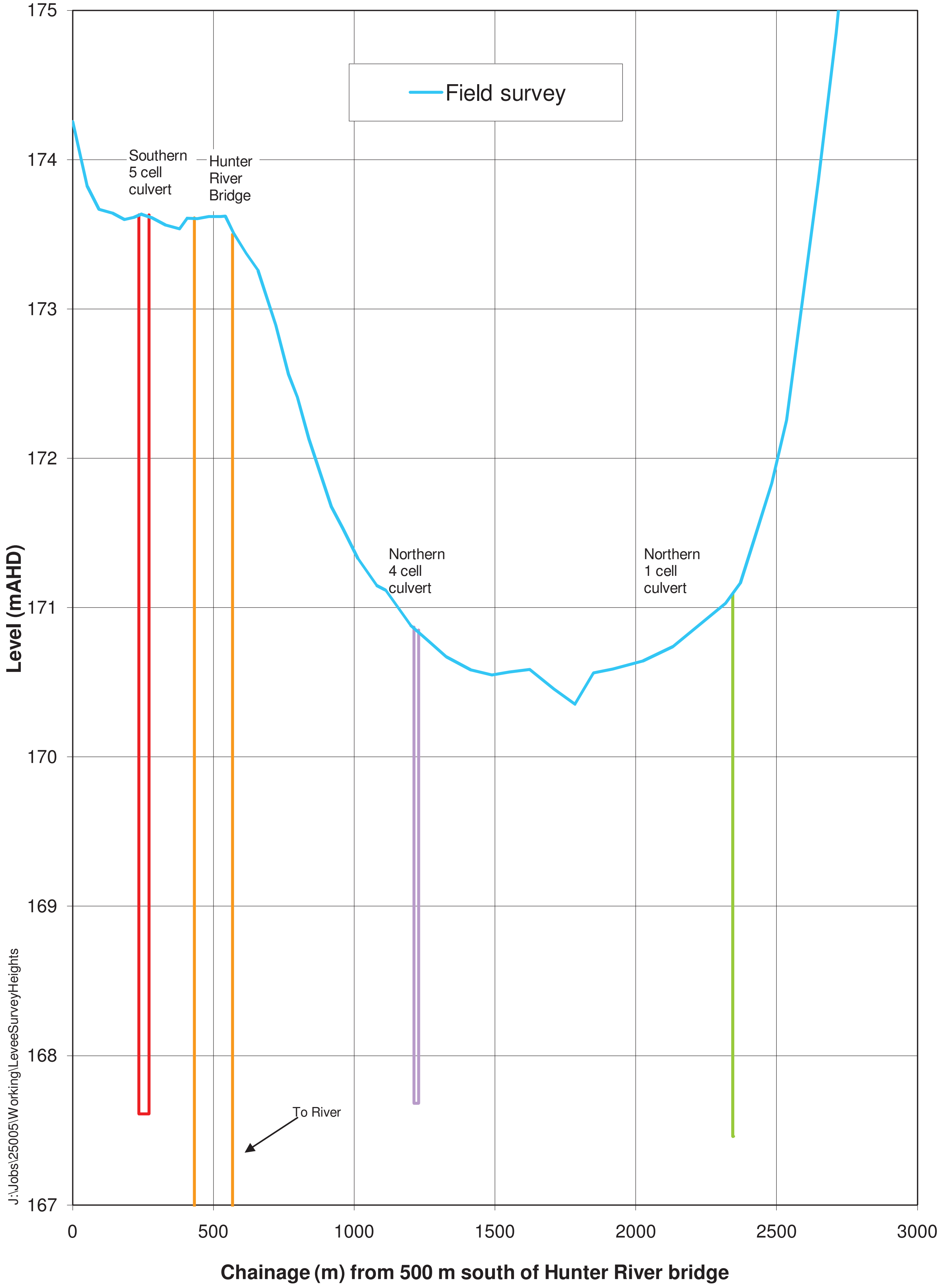

FIGURE 9
HYDRAULIC MODEL LAYOUT AND RESULT LOCATIONS

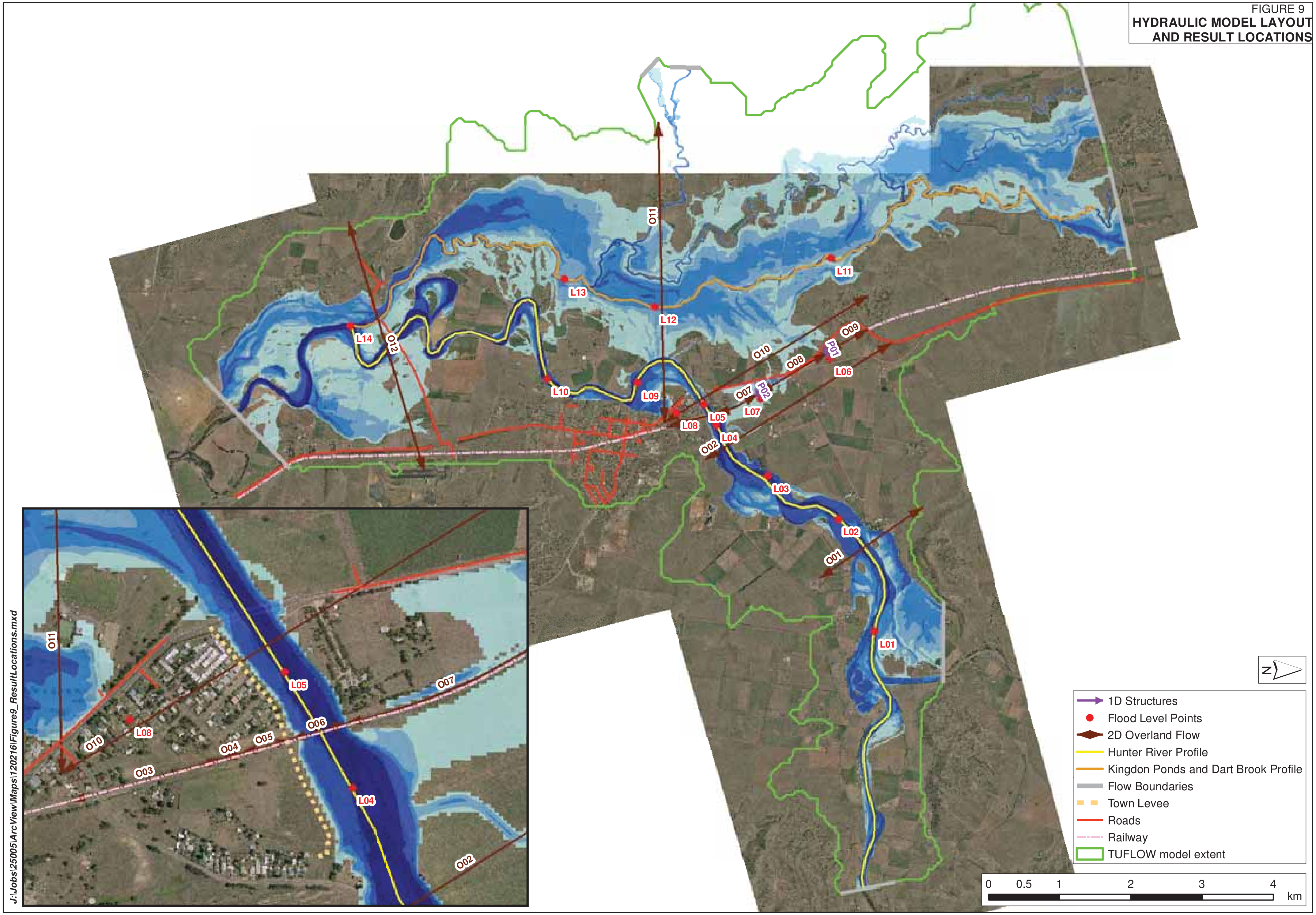

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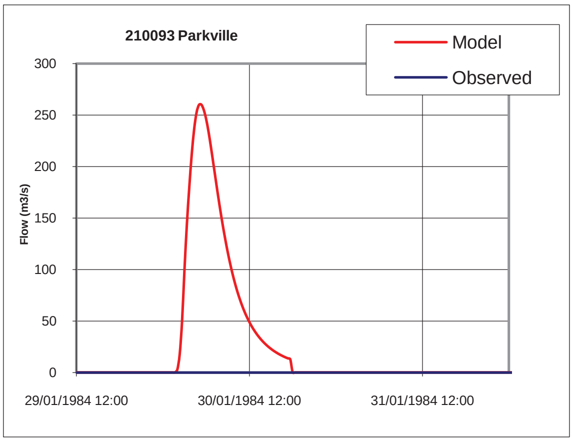
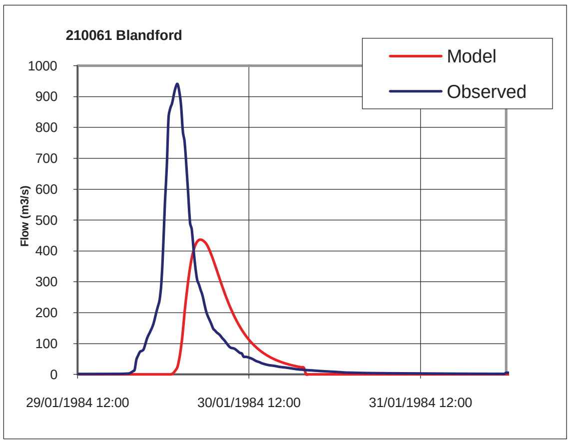
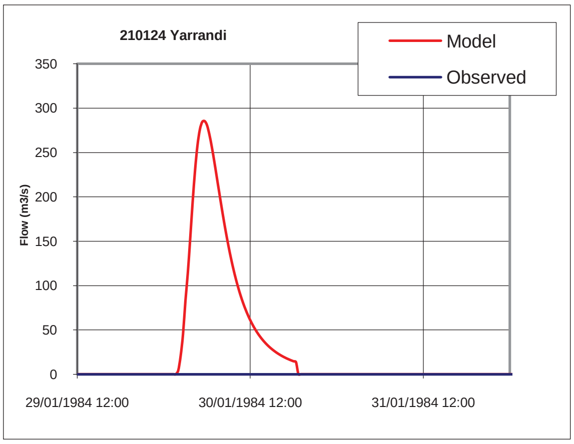
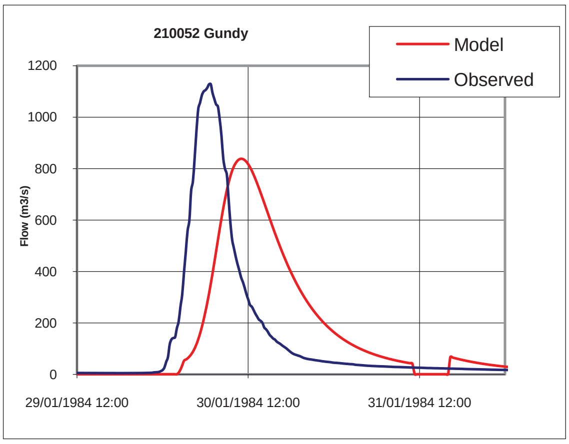
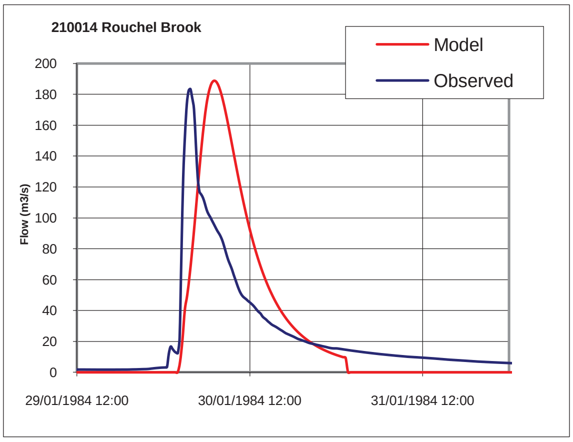
FIGURE 10
WBNM CALIBRATION JANUARY 1984

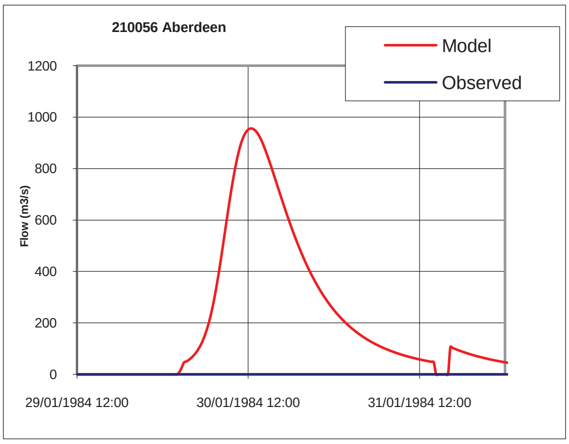












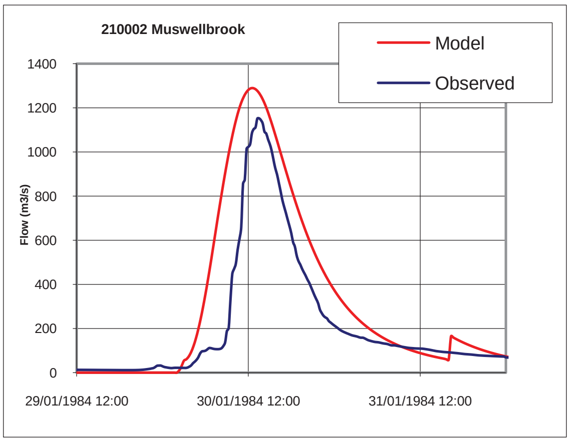


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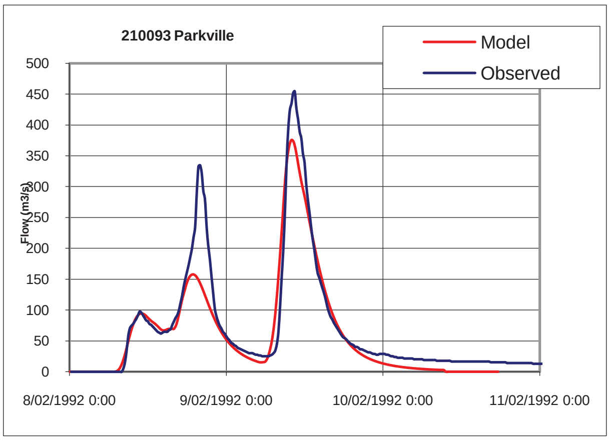
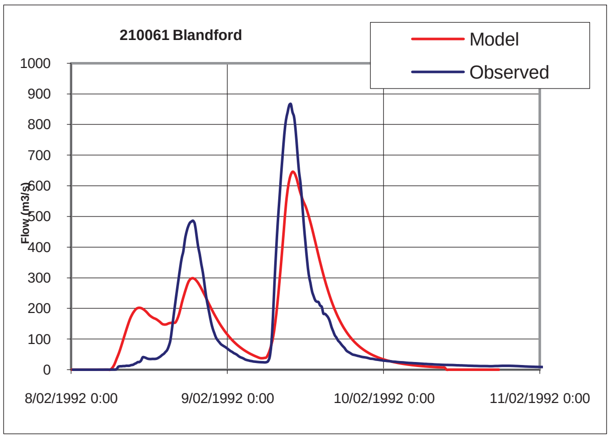
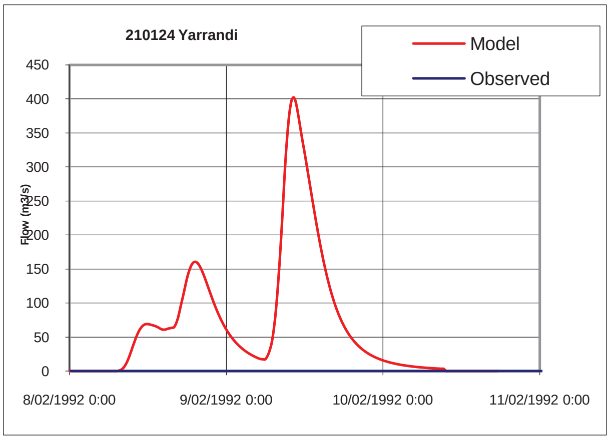
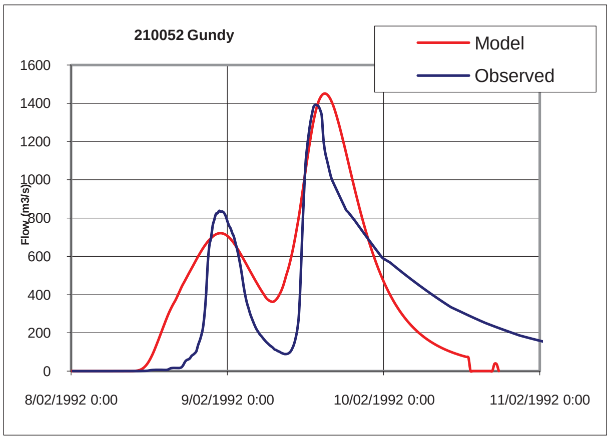
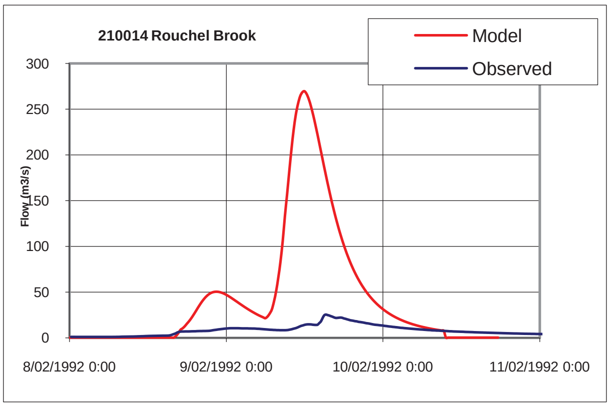
FIGURE 11
WBNM CALIBRATION
FEBRUARY 1992

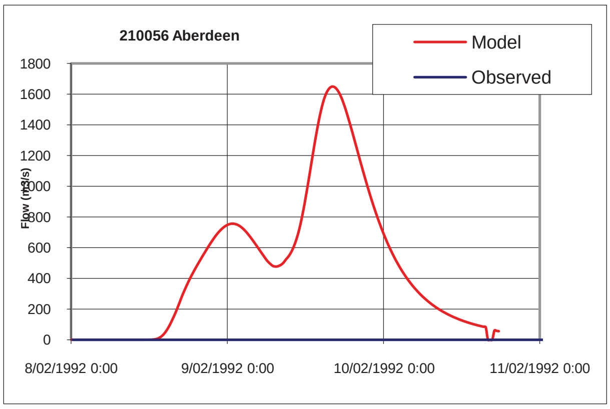












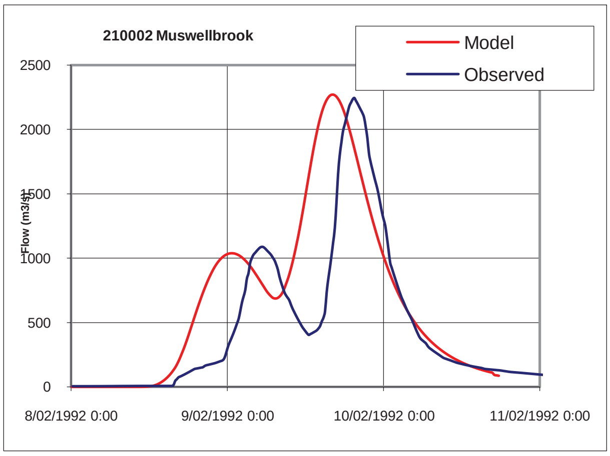


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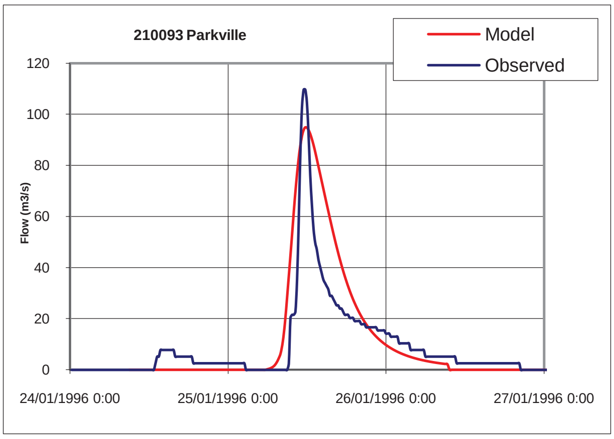
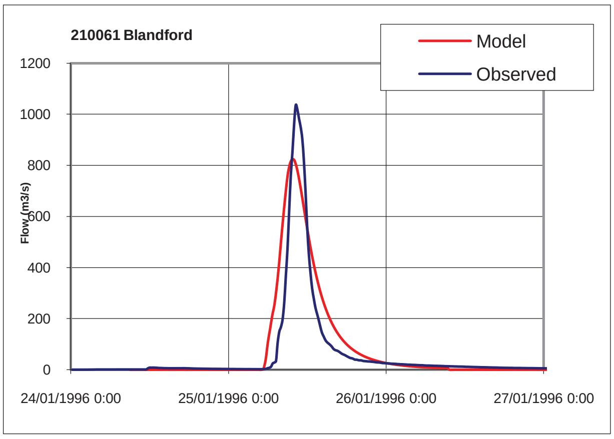
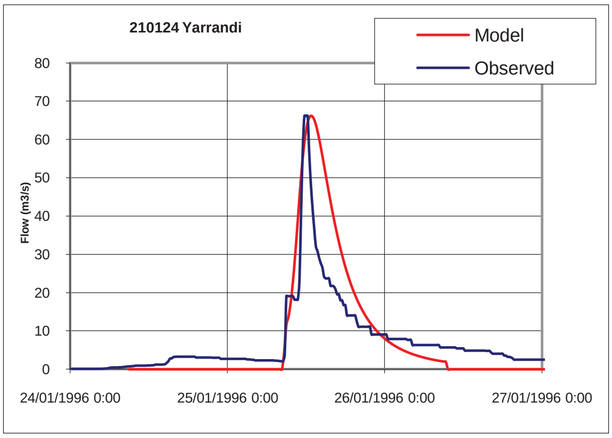
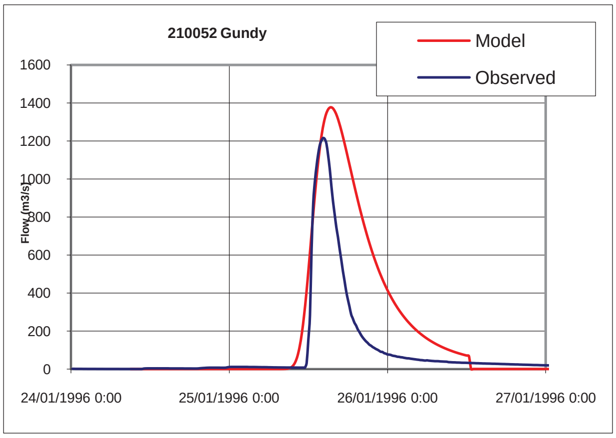
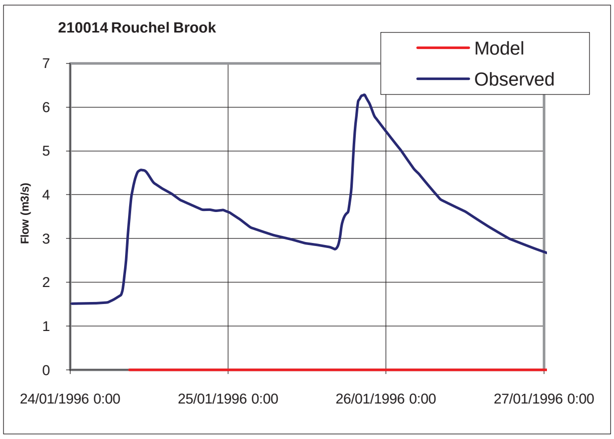
FIGURE 12
WBNM CALIBRATION JANUARY 1996

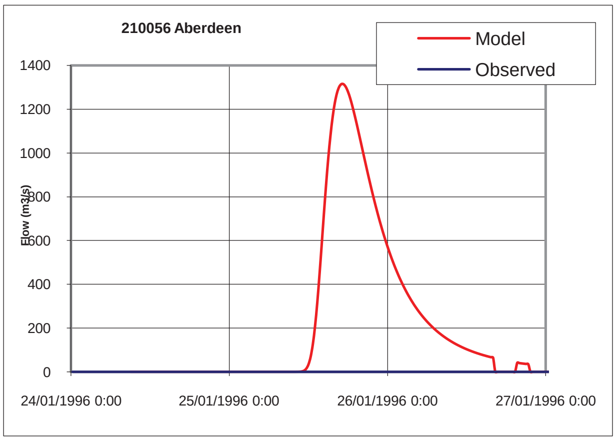












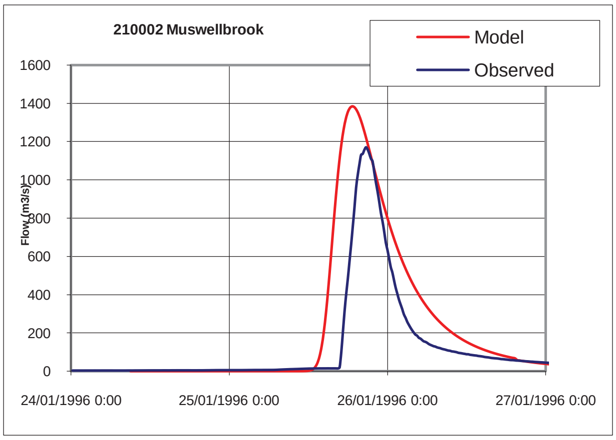


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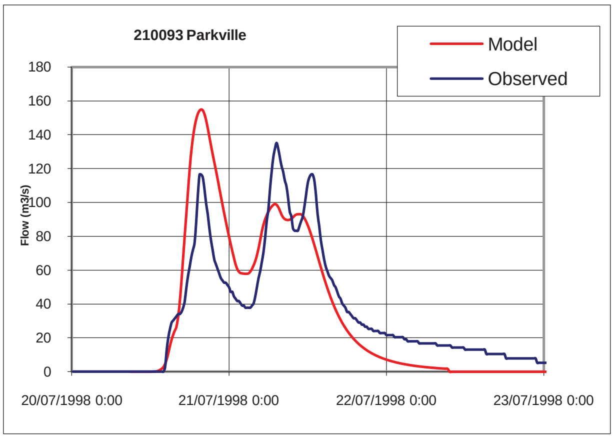
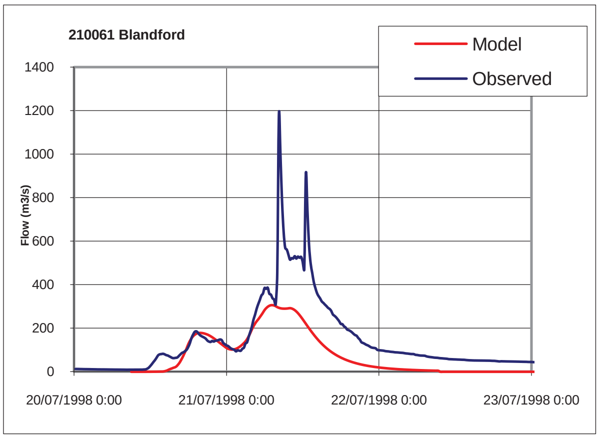
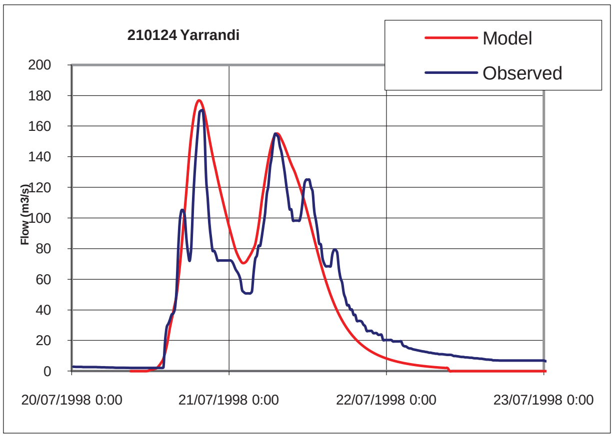
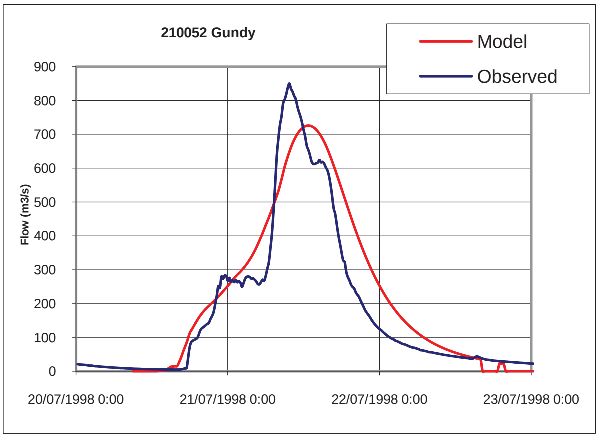
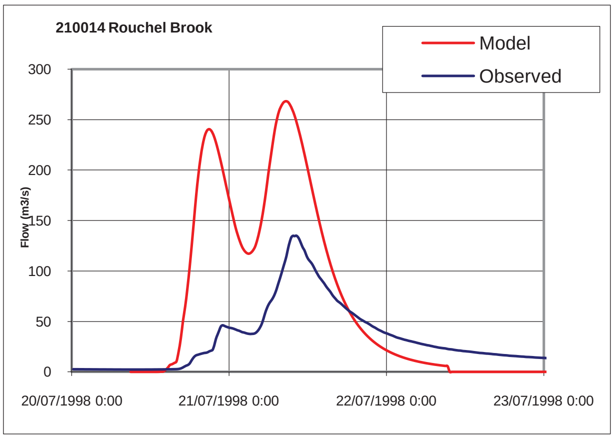
FIGURE 13
WBNM CALIBRATION
JULY 21ST 1998

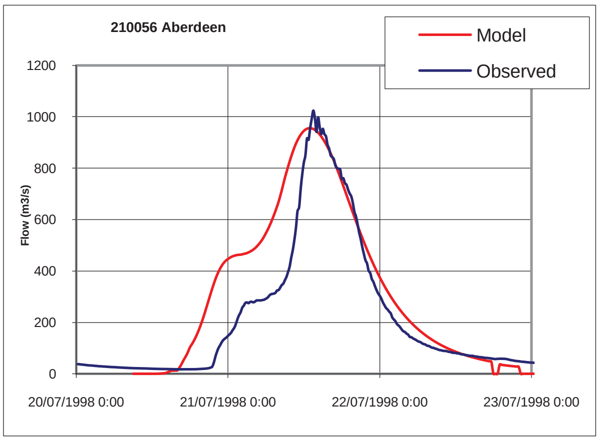












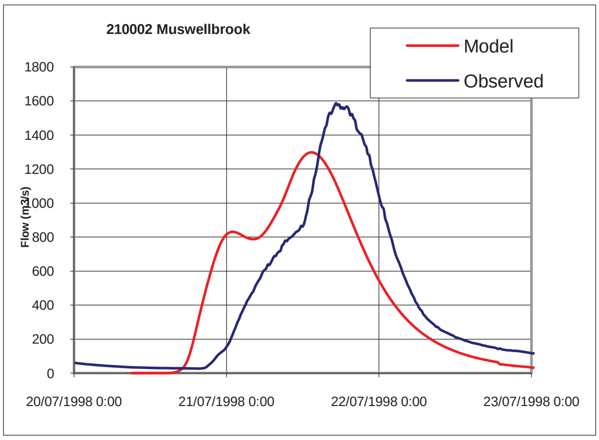


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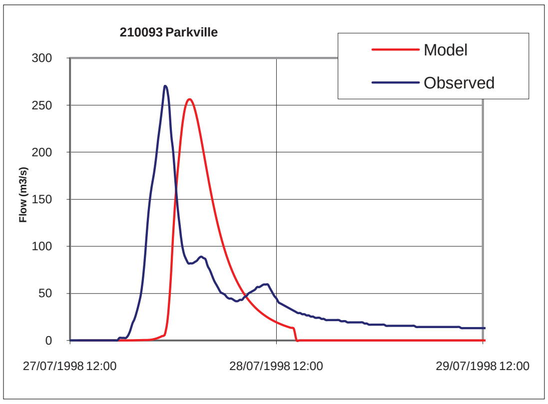
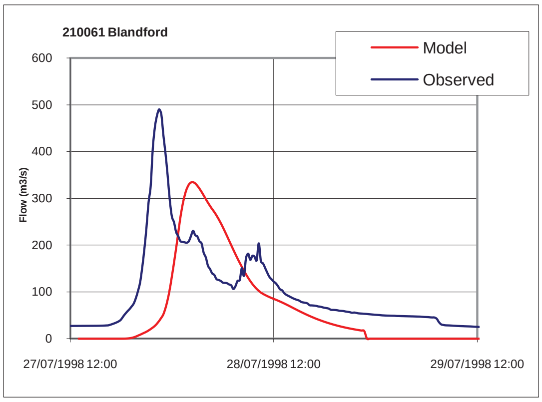
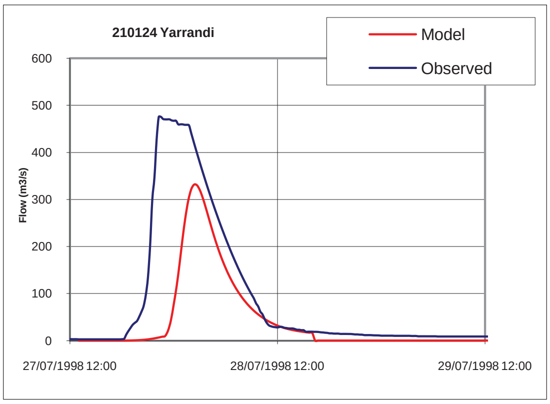
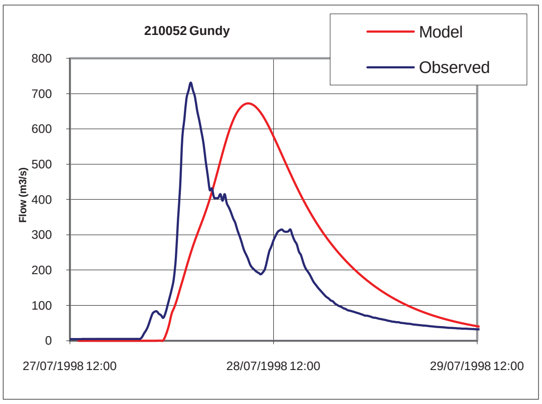
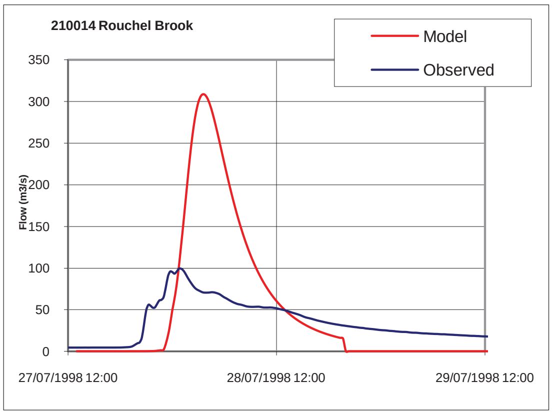
FIGURE 14
WBNM CALIBRATION
JULY 28TH 1998

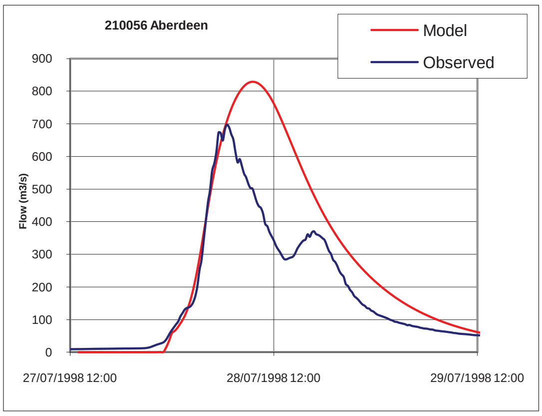












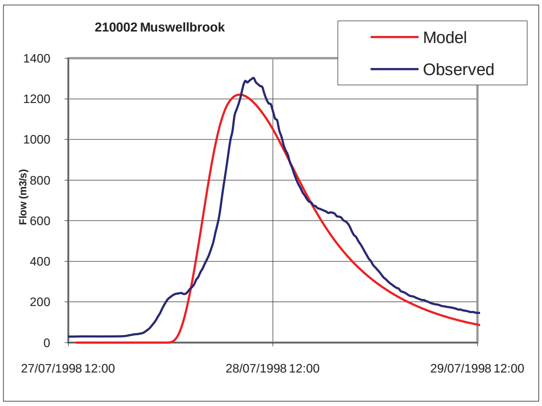


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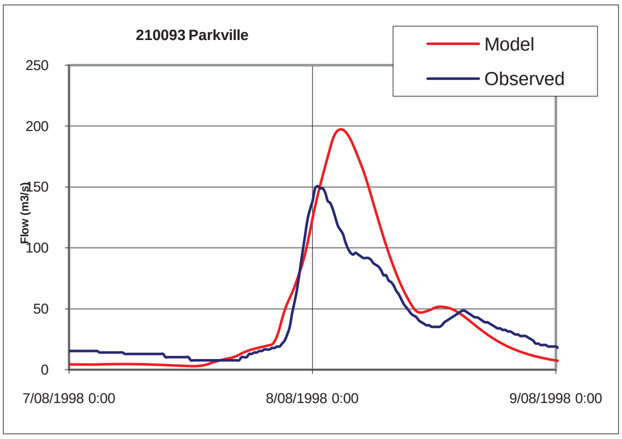
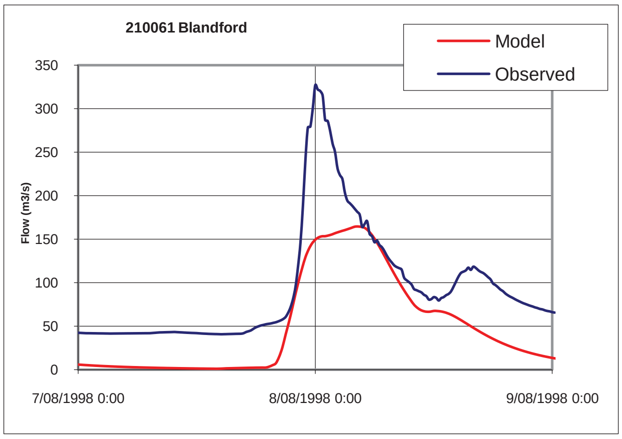
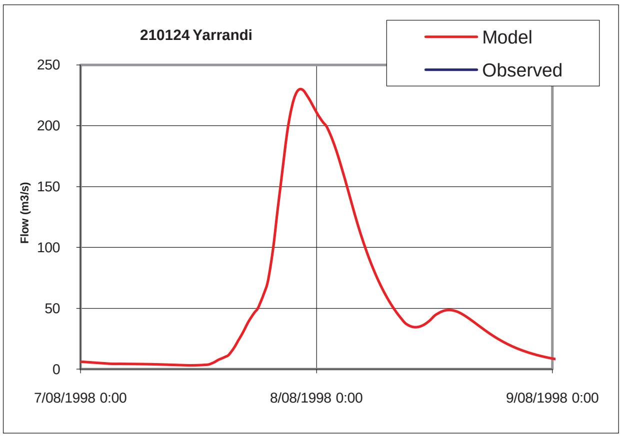
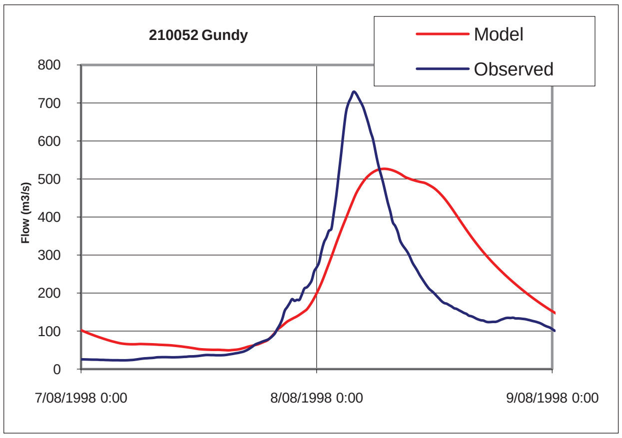
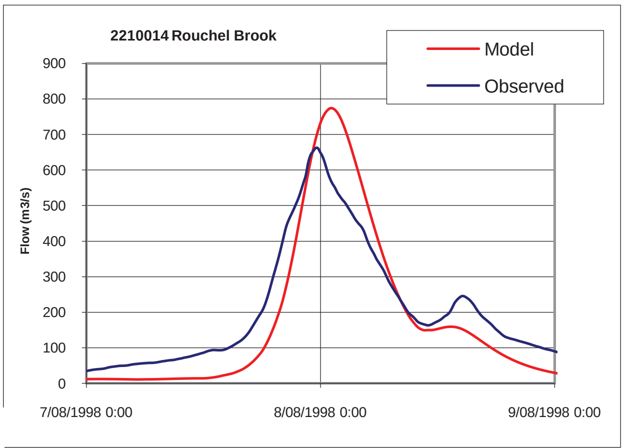
FIGURE 15
WBNM CALIBRATION
AUGUST 1998

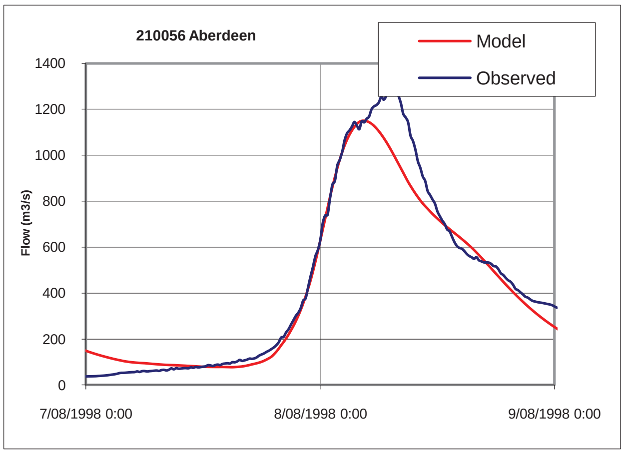












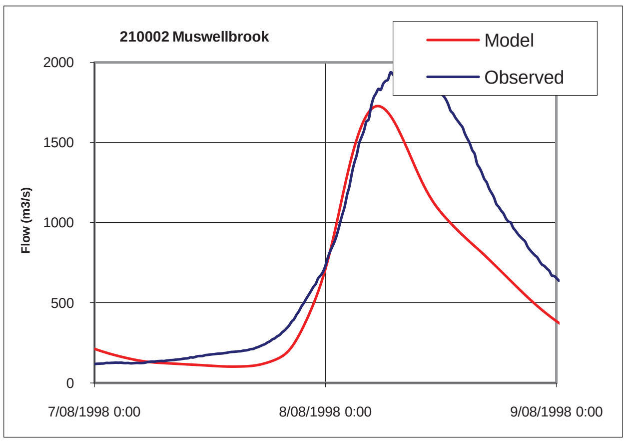


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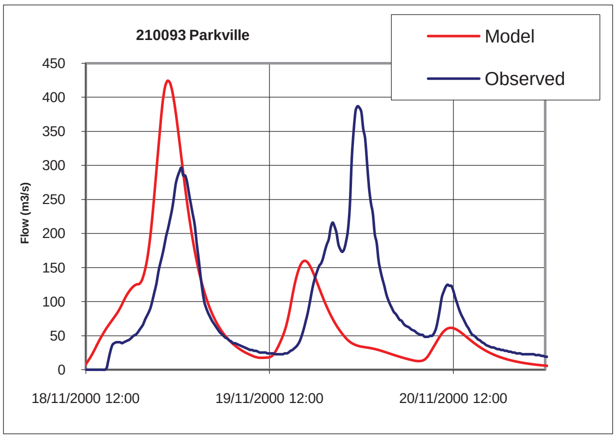
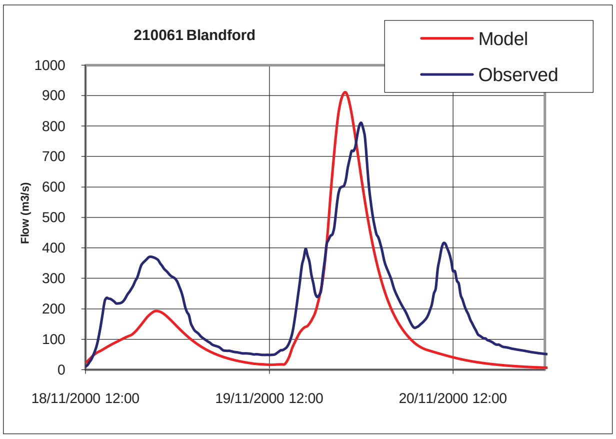
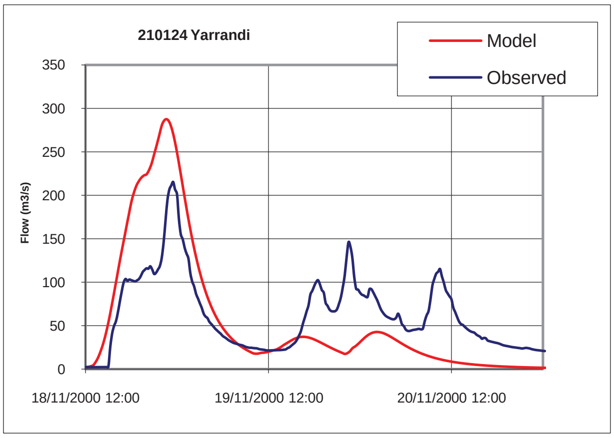
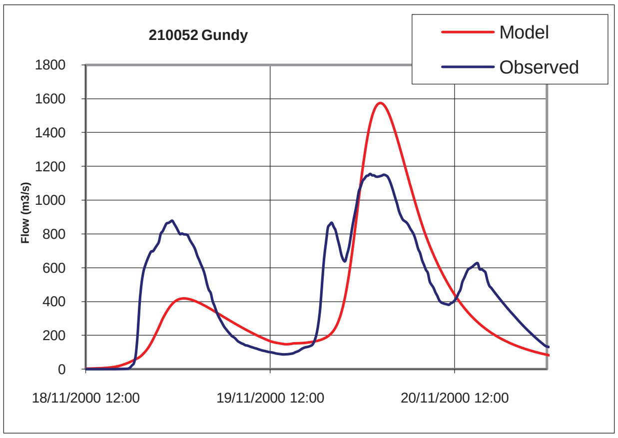
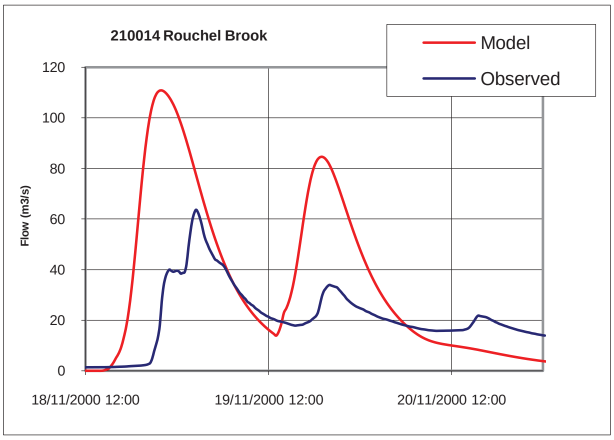
FIGURE 16
WBNM CALIBRATION
NOVEMBER 2000

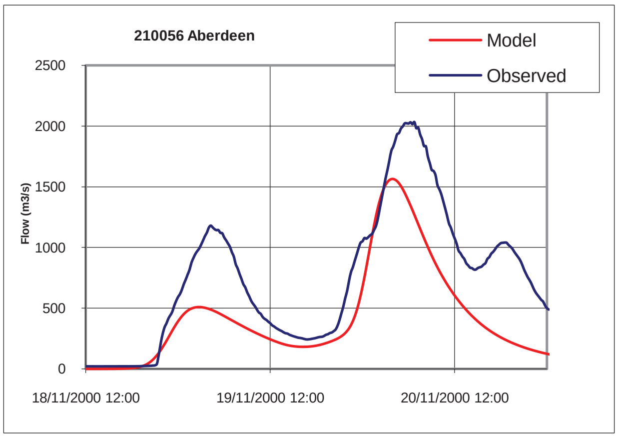












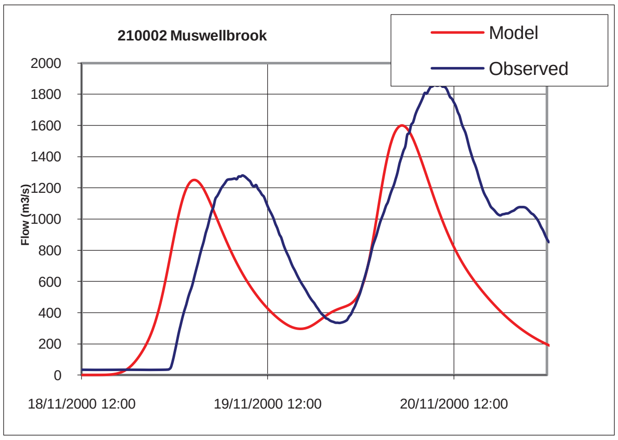


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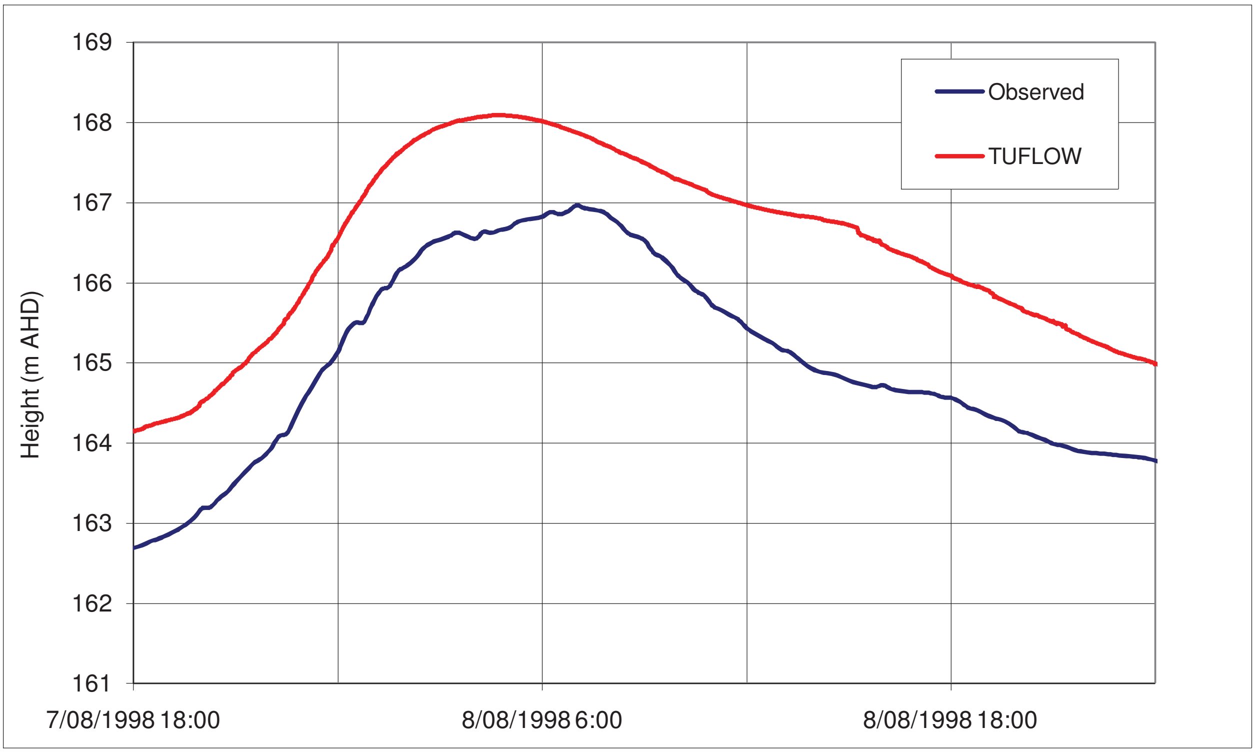
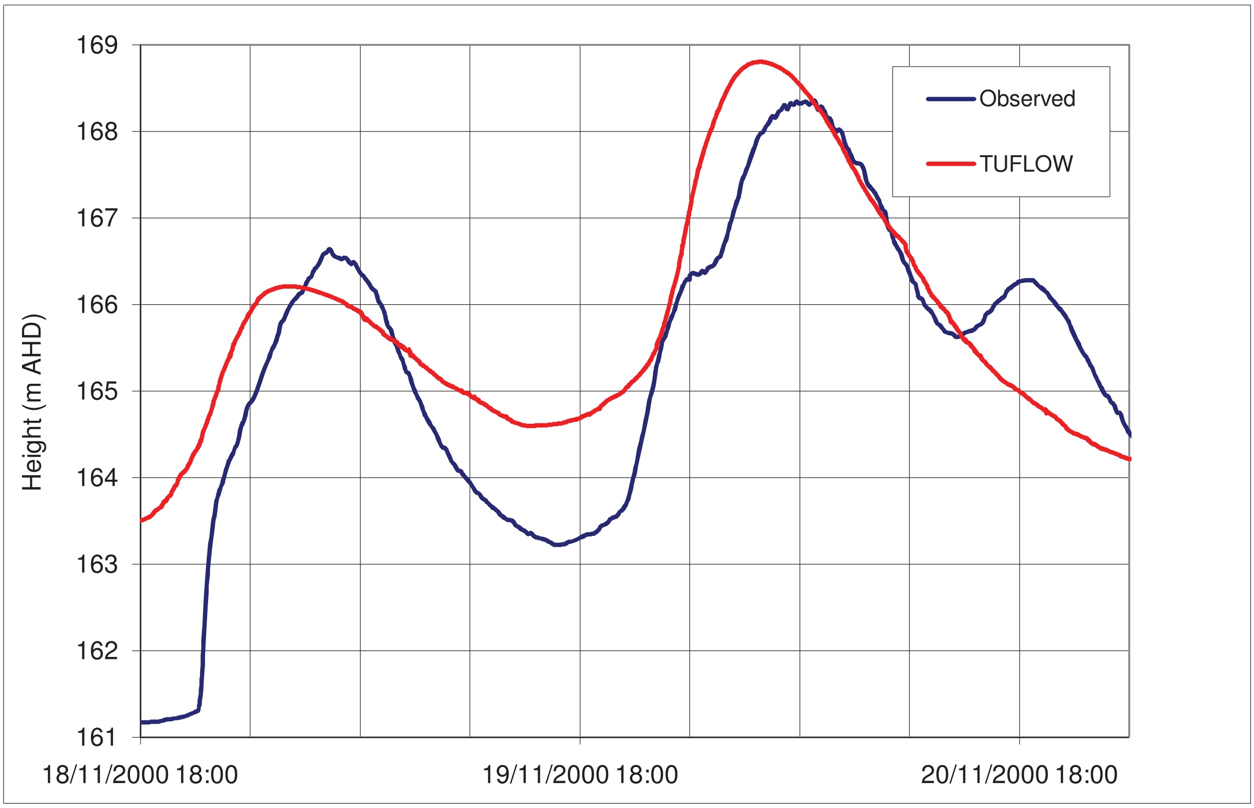
FIGURE 17
STAGE HYDROGRAPHS AT ABERDEEN





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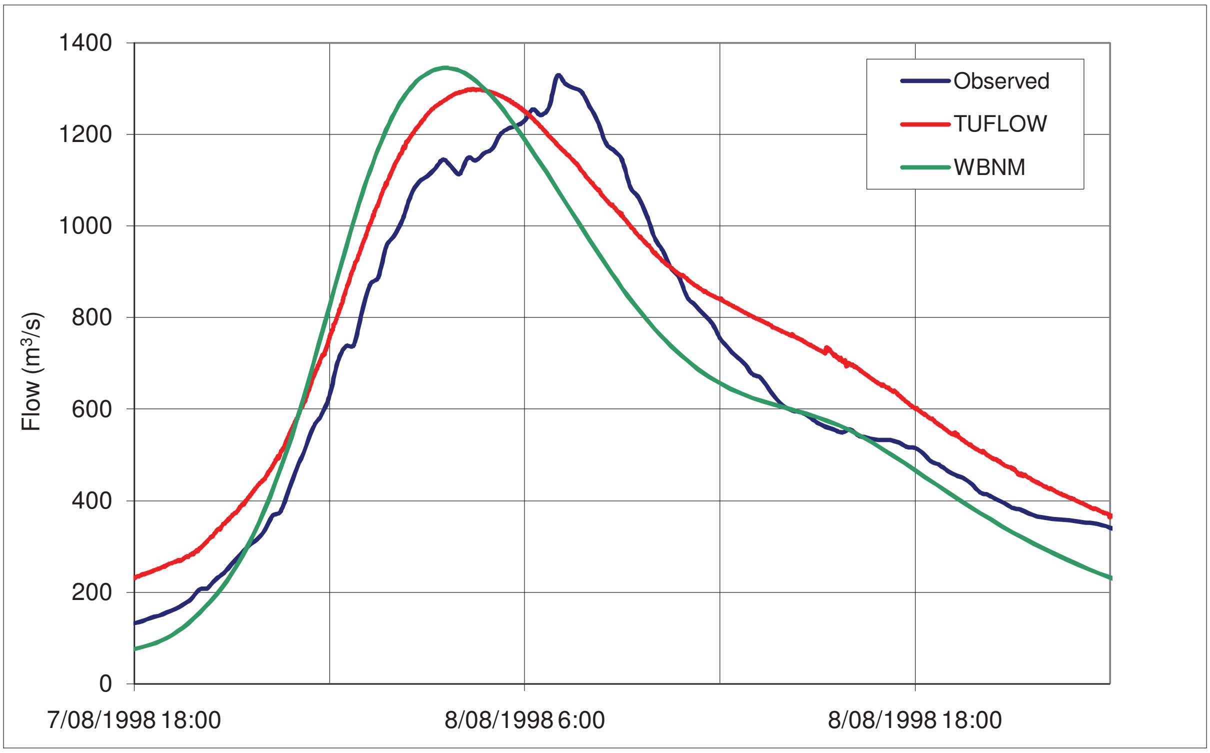
FLOW HYDROGRAPHS AT ABERDEEN



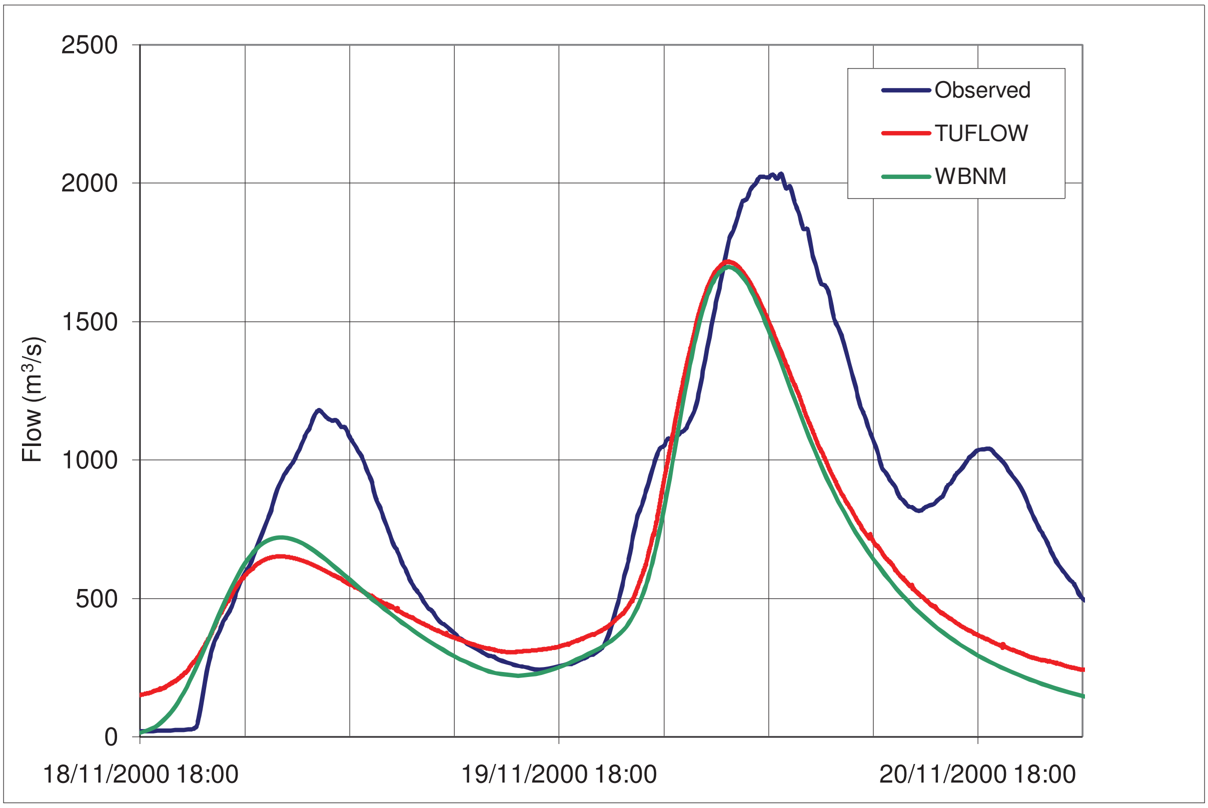


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FIGURE 19
HUNTER RIVER
DESIGN FLOOD PROFILES

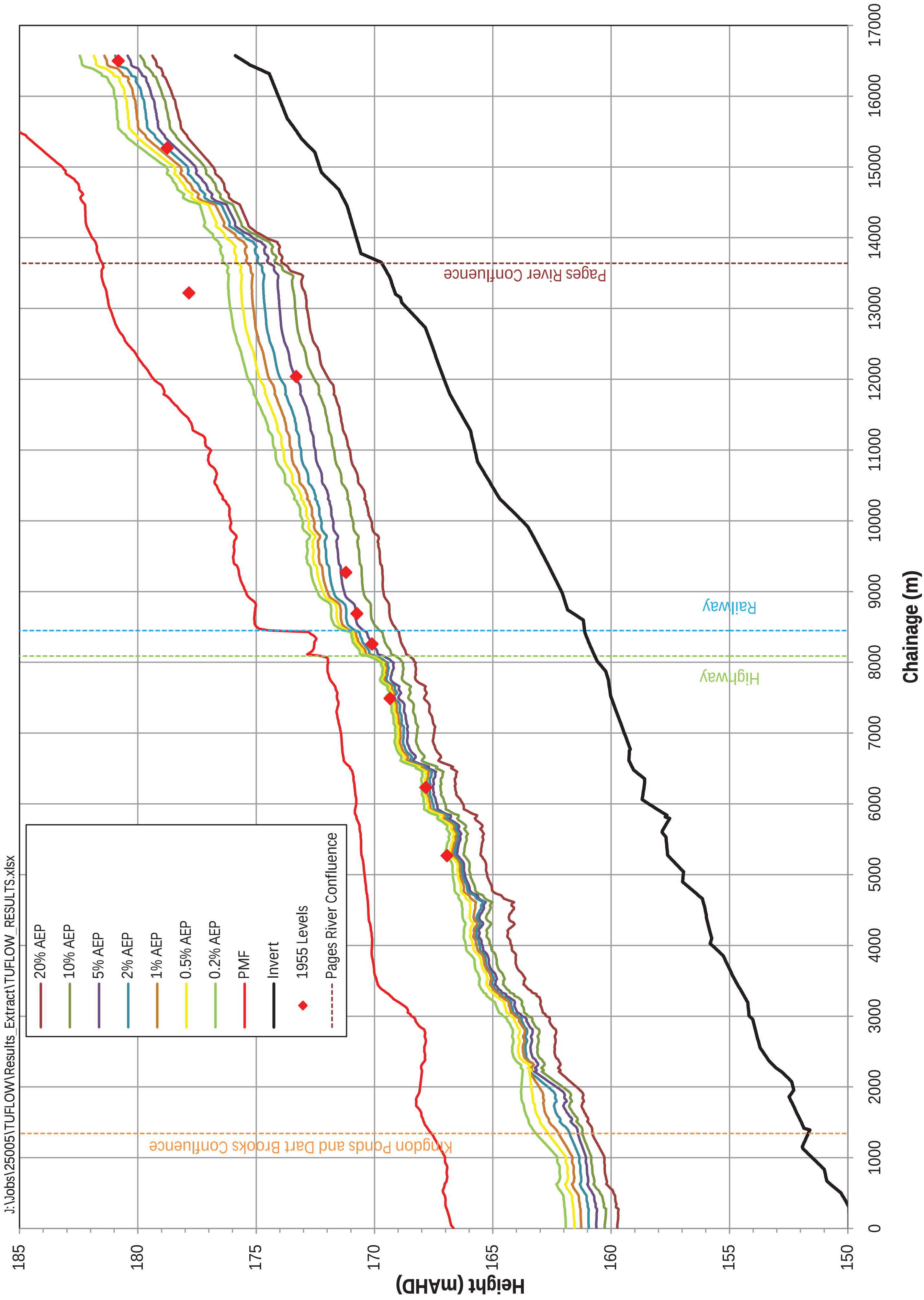

FIGURE 20

KINGDON PONDS AND DART BROOK DESIGN FLOOD PROFILES

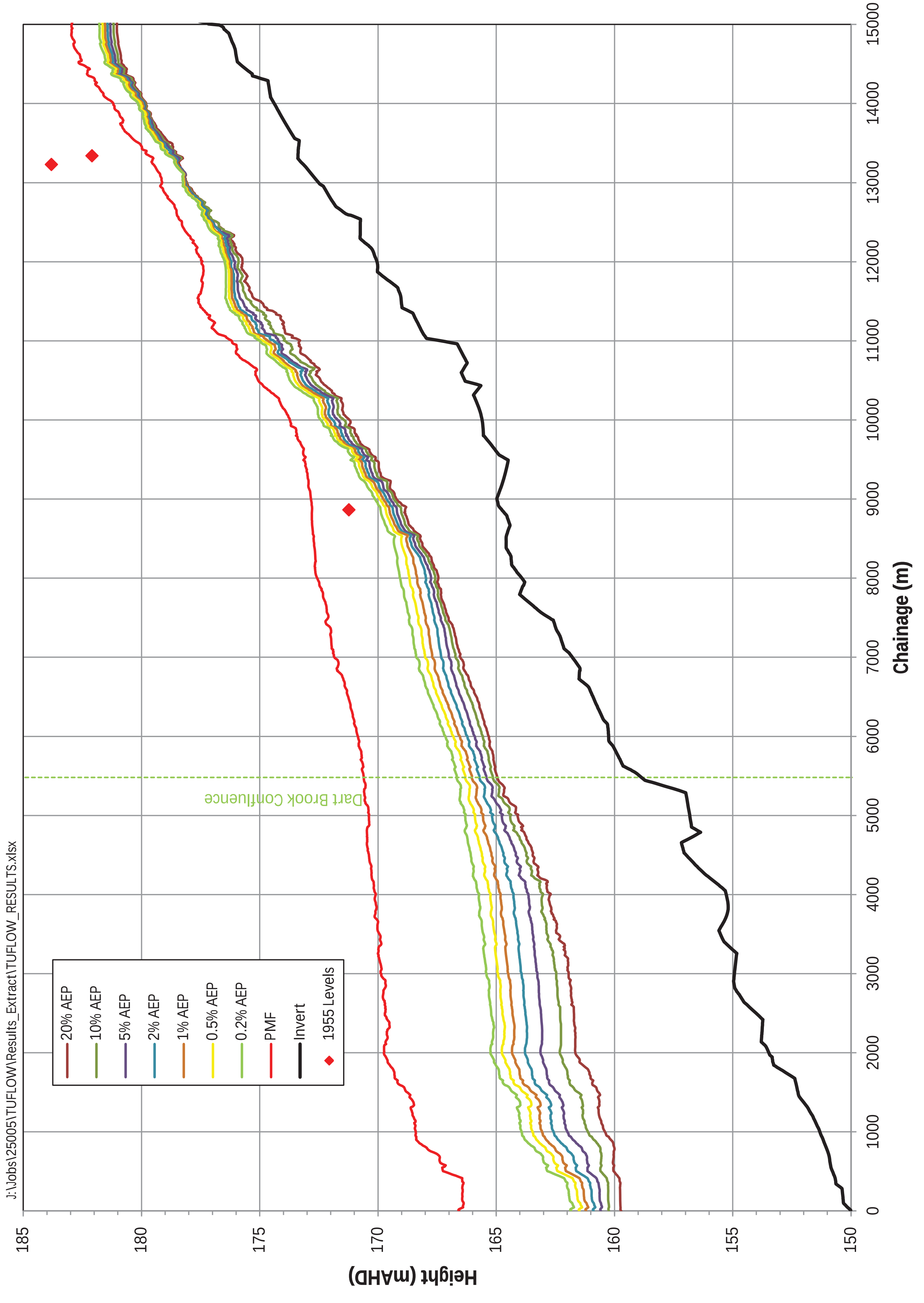

FIGURE 21
20% AEP (5 YR ARI)
FLOOD DEPTHS AND CONTOURS

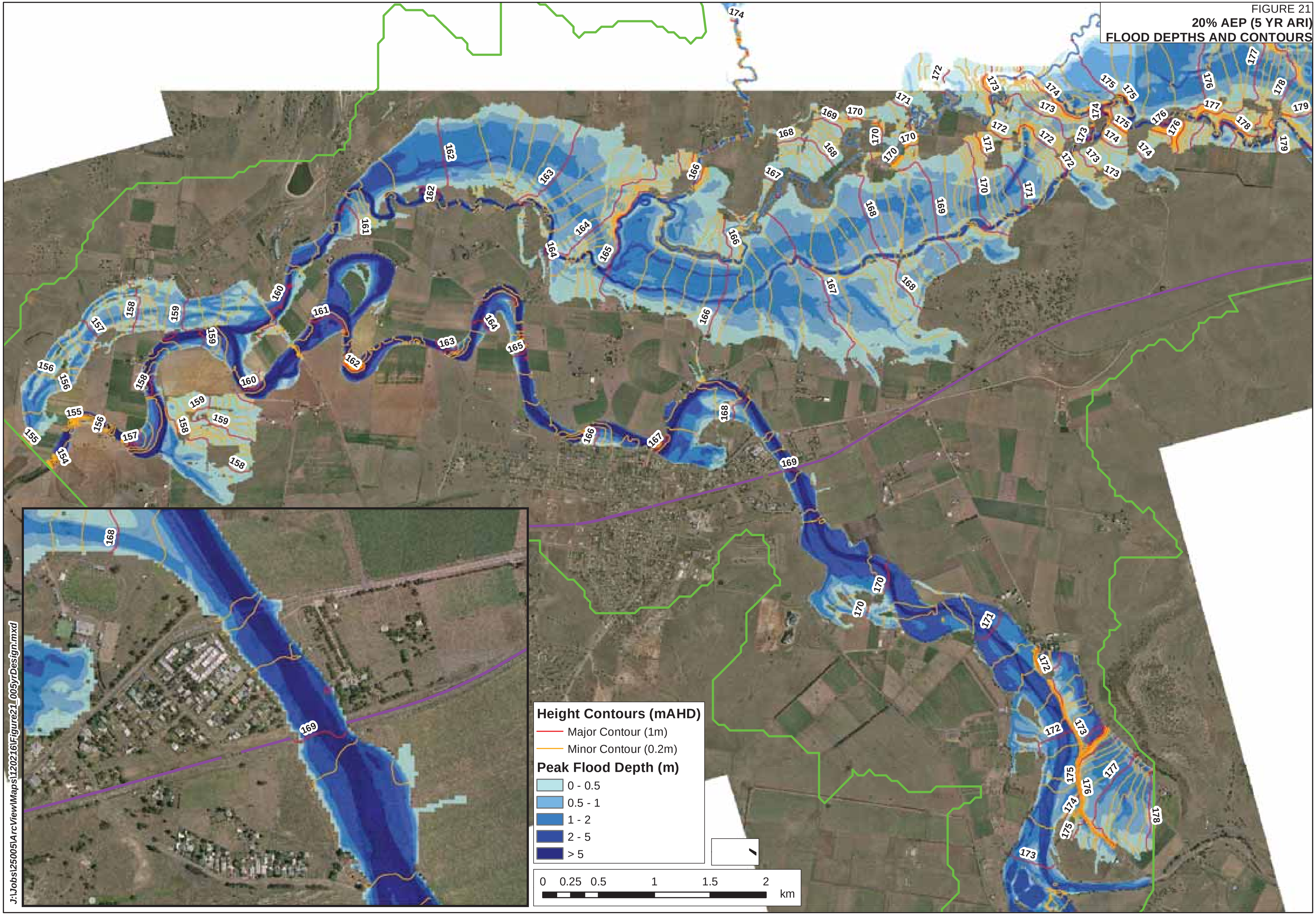

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FIGURE 22
10% AEP (10 YR ARI)
FLOOD DEPTHS AND CONTOURS

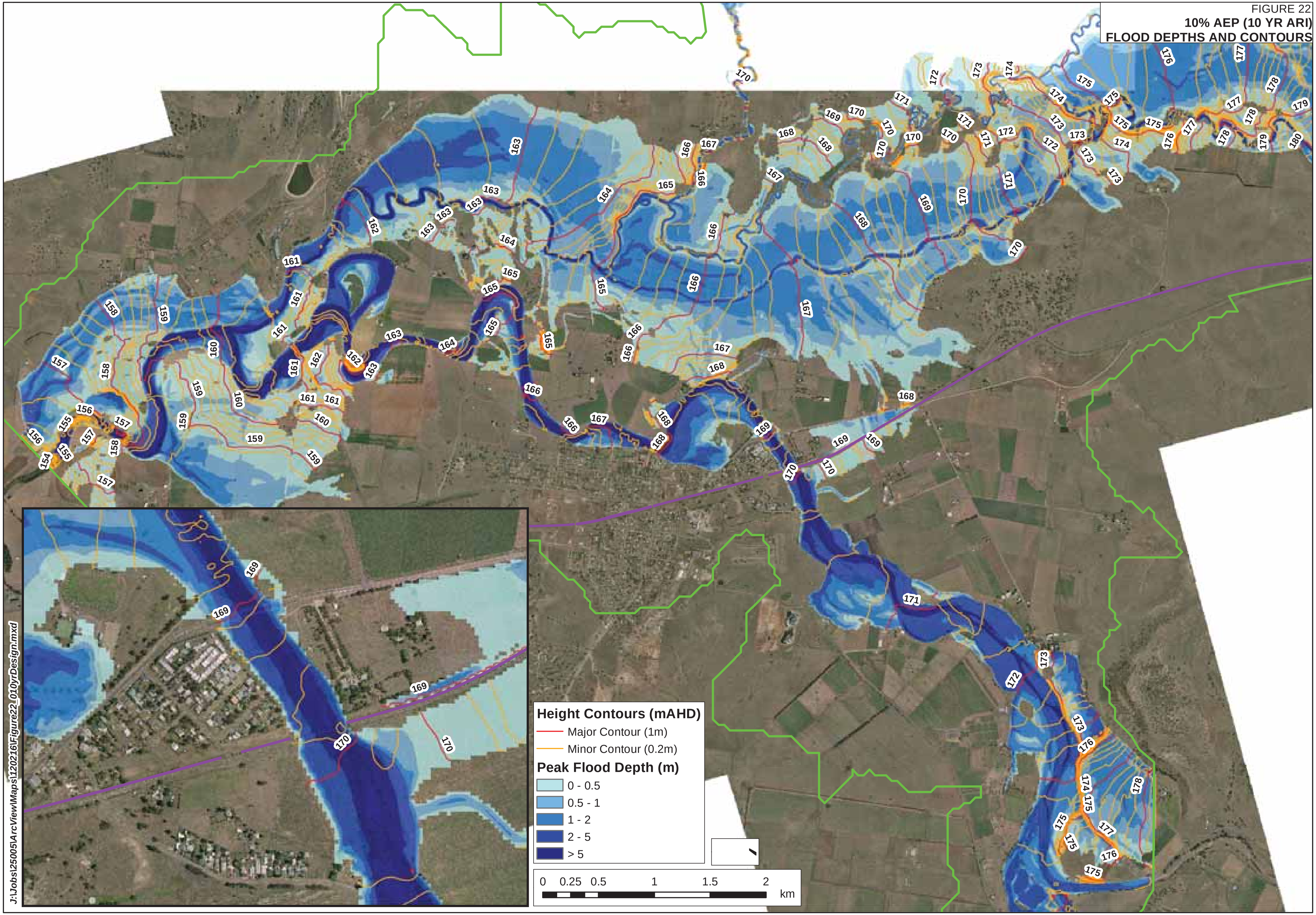

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FIGURE 23
5% AEP (20 YR ARI)
FLOOD DEPTHS AND CONTOURS

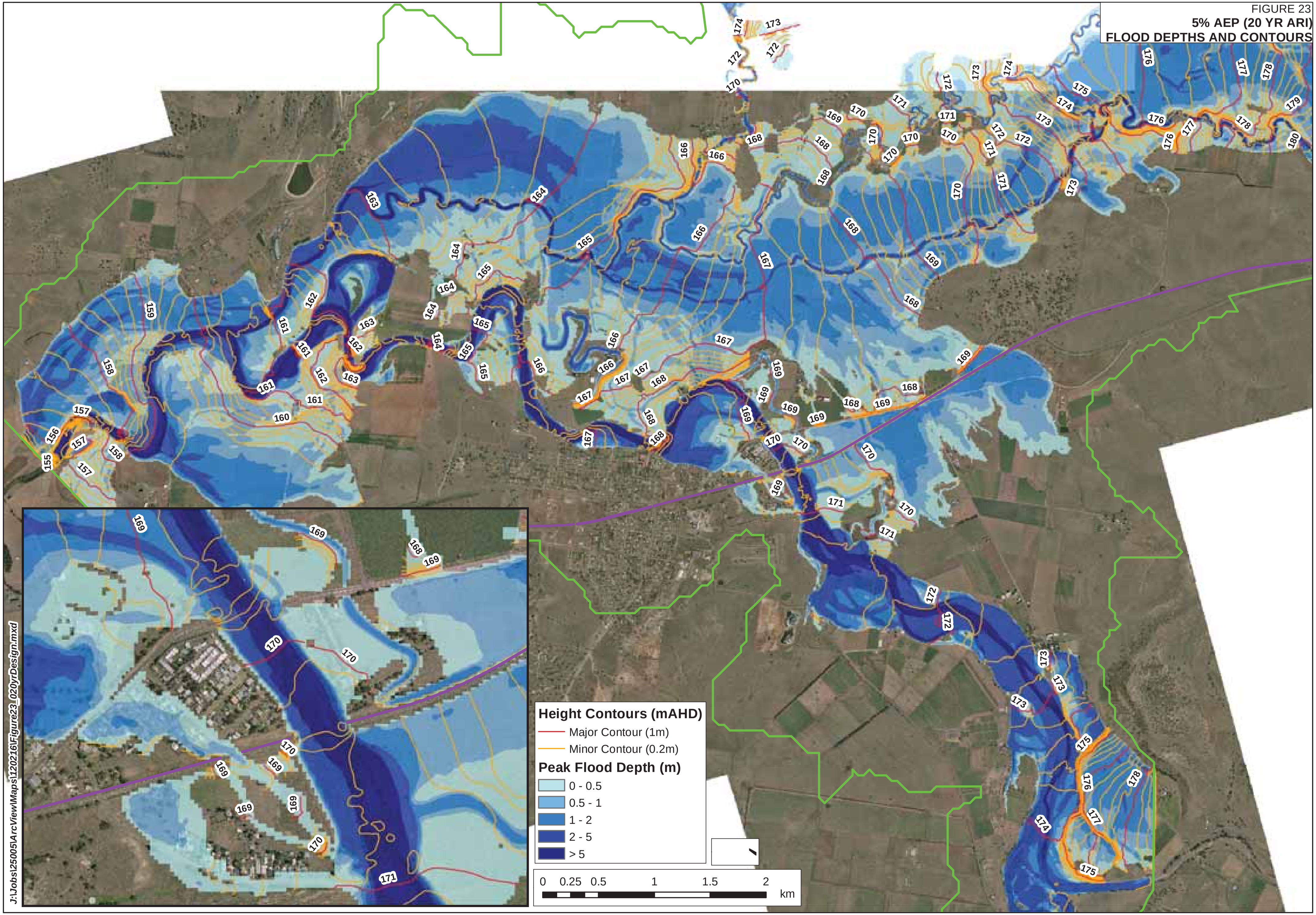

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FIGURE 24
2% AEP (50 YR ARI)
FLOOD DEPTHS AND CONTOURS

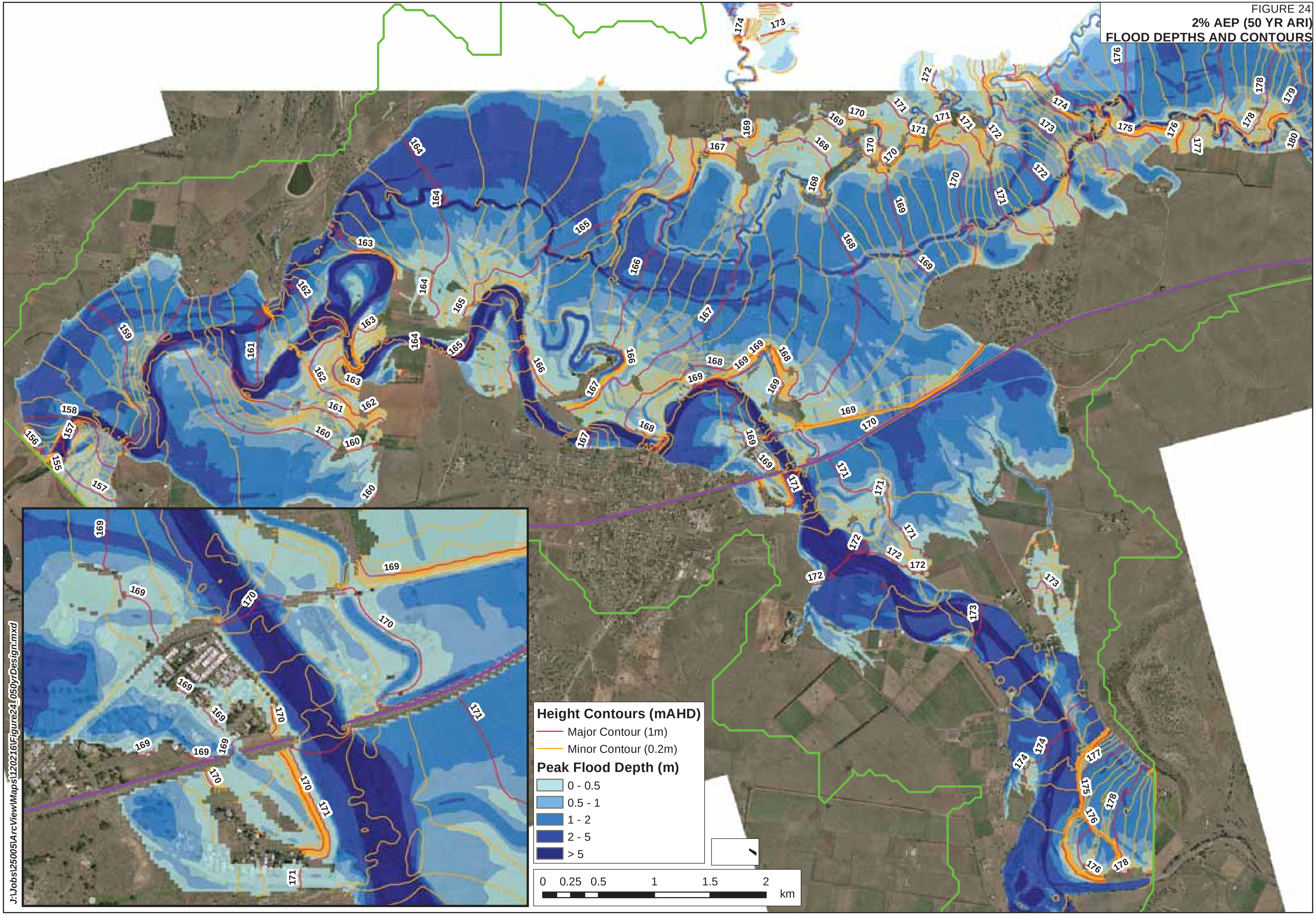

FIGURE 25
1% AEP (100 YR ARI)
FLOOD DEPTHS AND CONTOURS

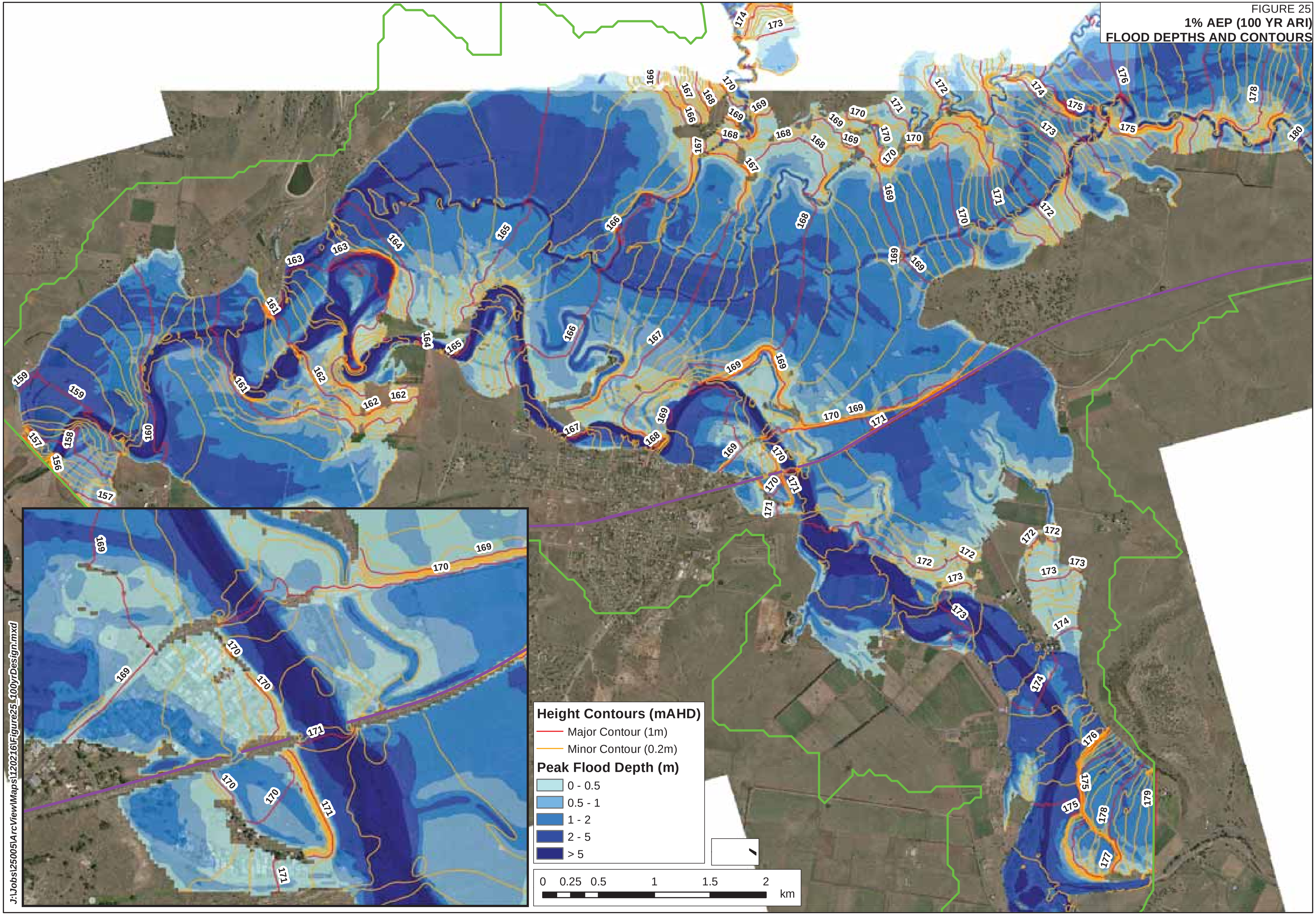

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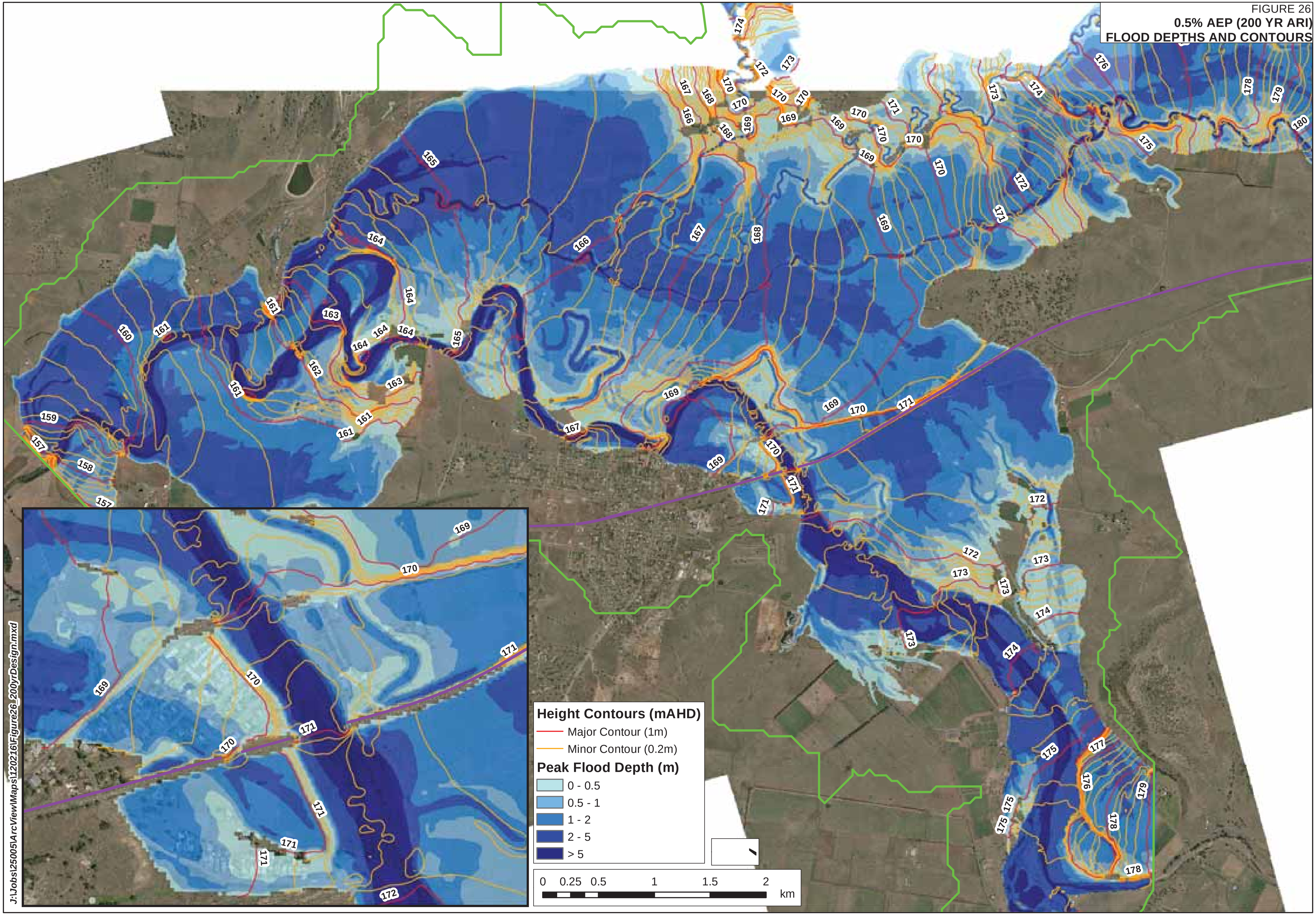
FIGURE 26
0.5% AEP (200 YR ARI) FLOOD DEPTHS AND CONTOURS



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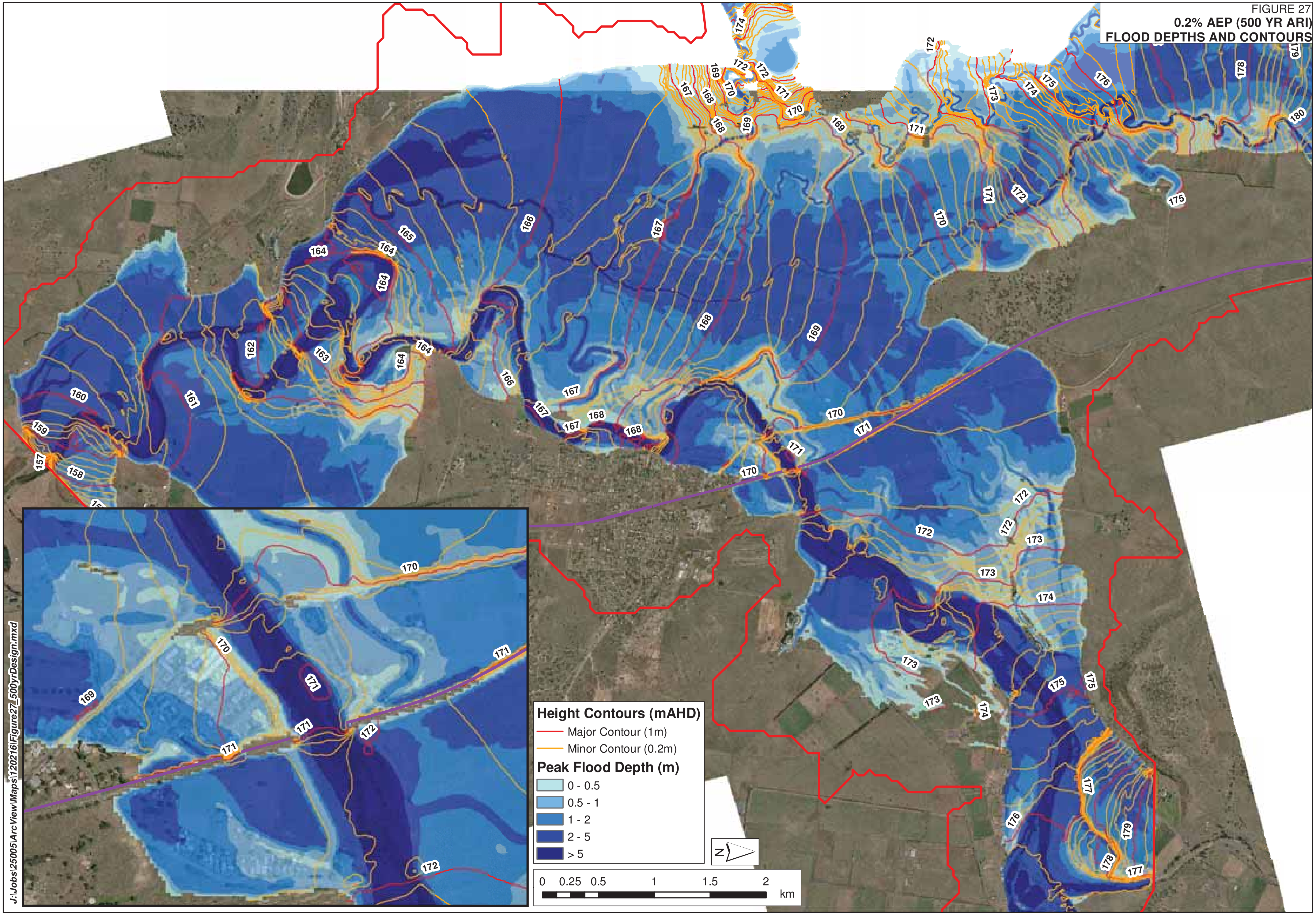

FIGURE 27
0.2% AEP (500 YR ARI)
FLOOD DEPTHS AND CONTOURS

FIGURE 28
PMF (24HR GTSMR) FLOOD DEPTHS AND CONTOURS

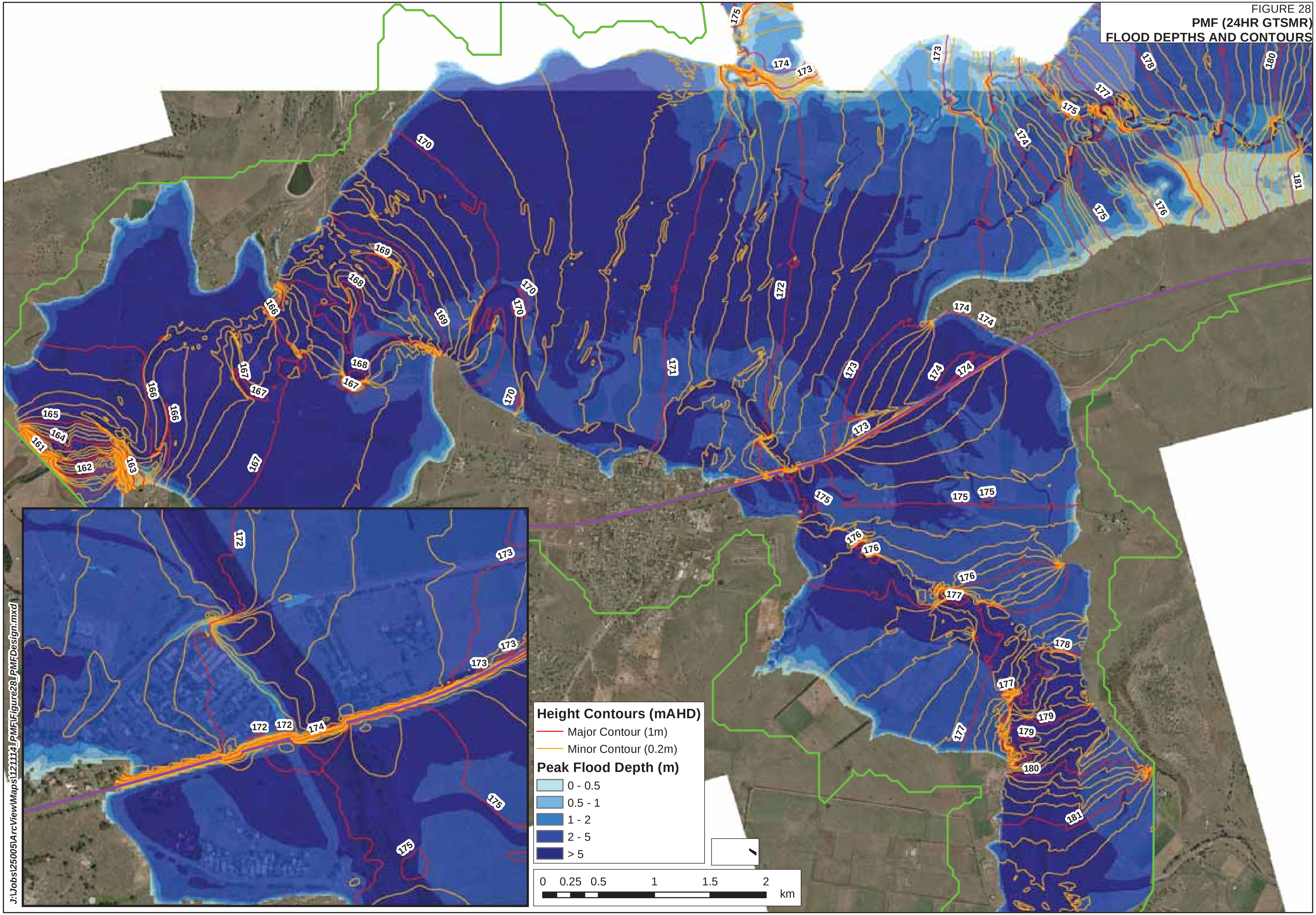

FIGURE 29
5% AEP (20 YR ARI)
PROVISIONAL HAZARD

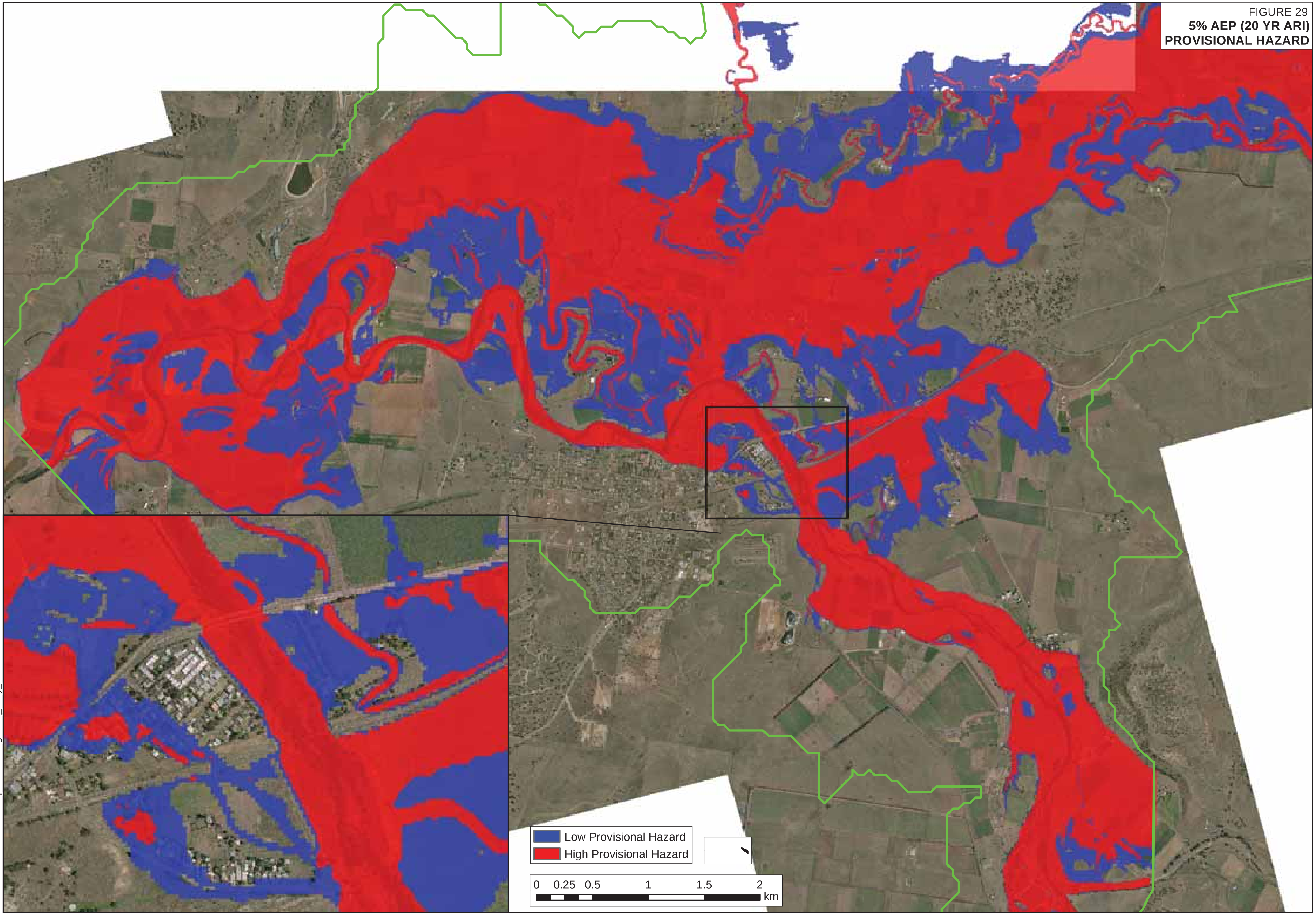

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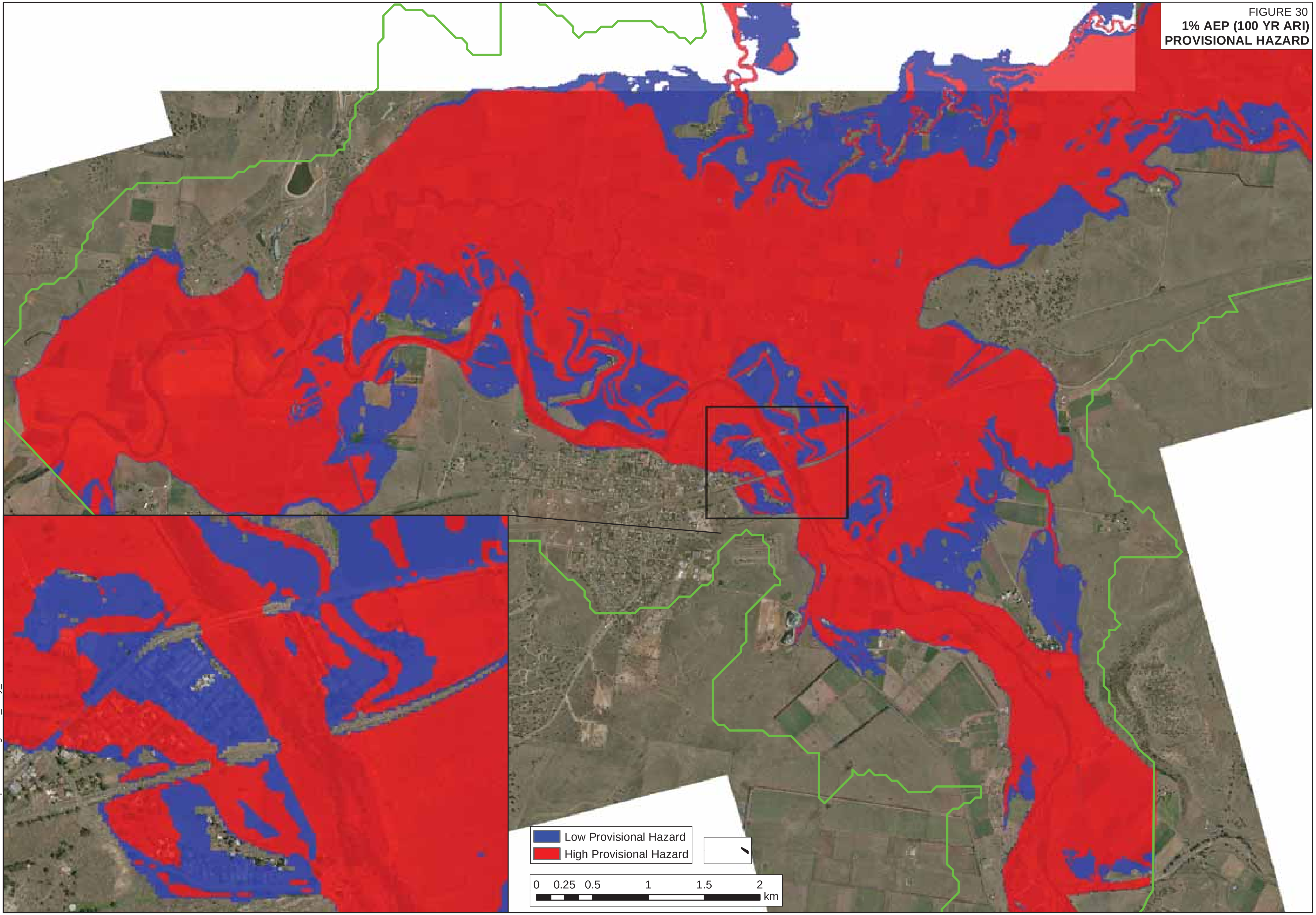
FIGURE 30
1% AEP (100 YR ARI)
PROVISIONAL HAZARD



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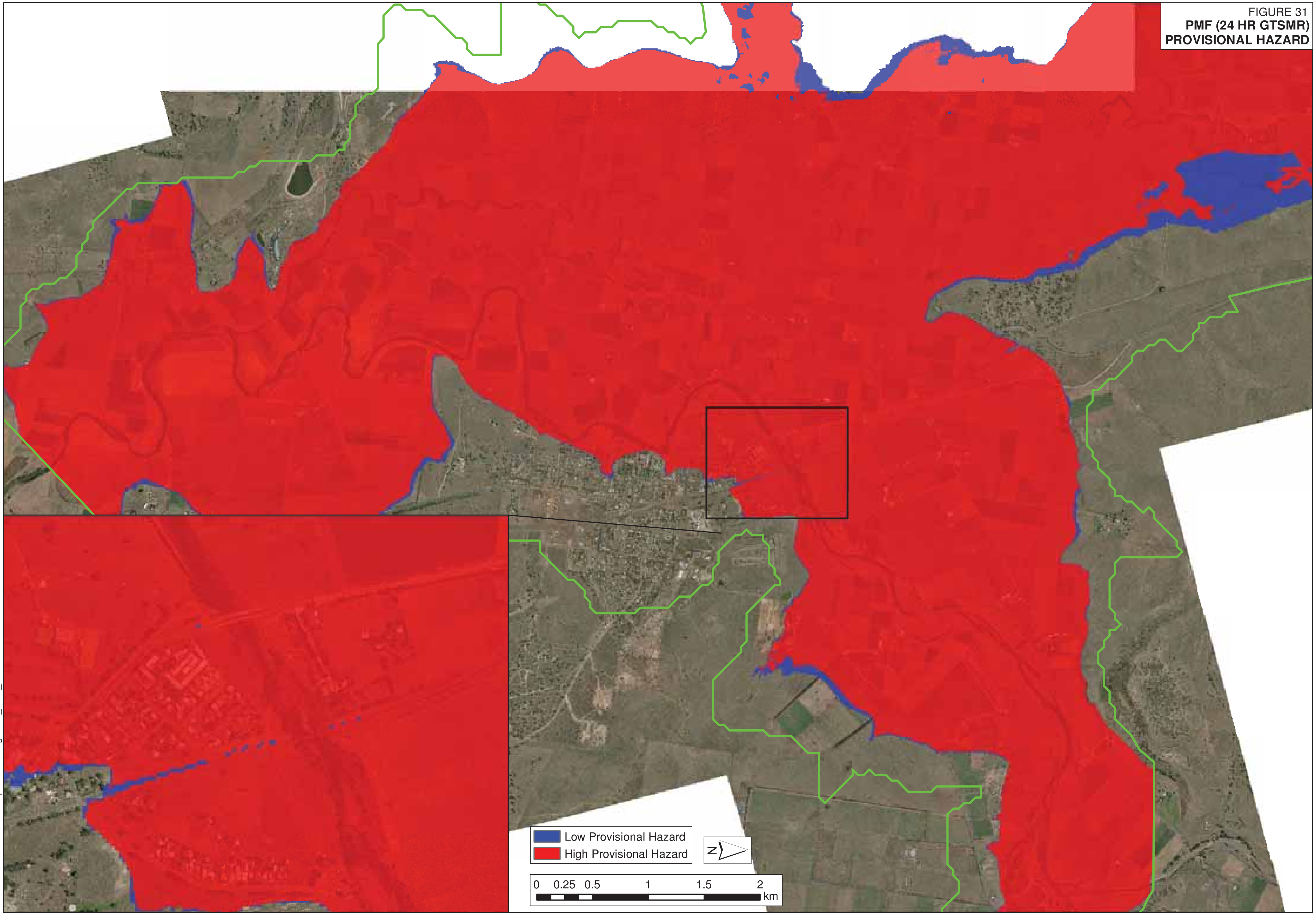

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FIGURE 32
5% AEP (20 YR ARI)
HYDRAULIC CATEGORISATION

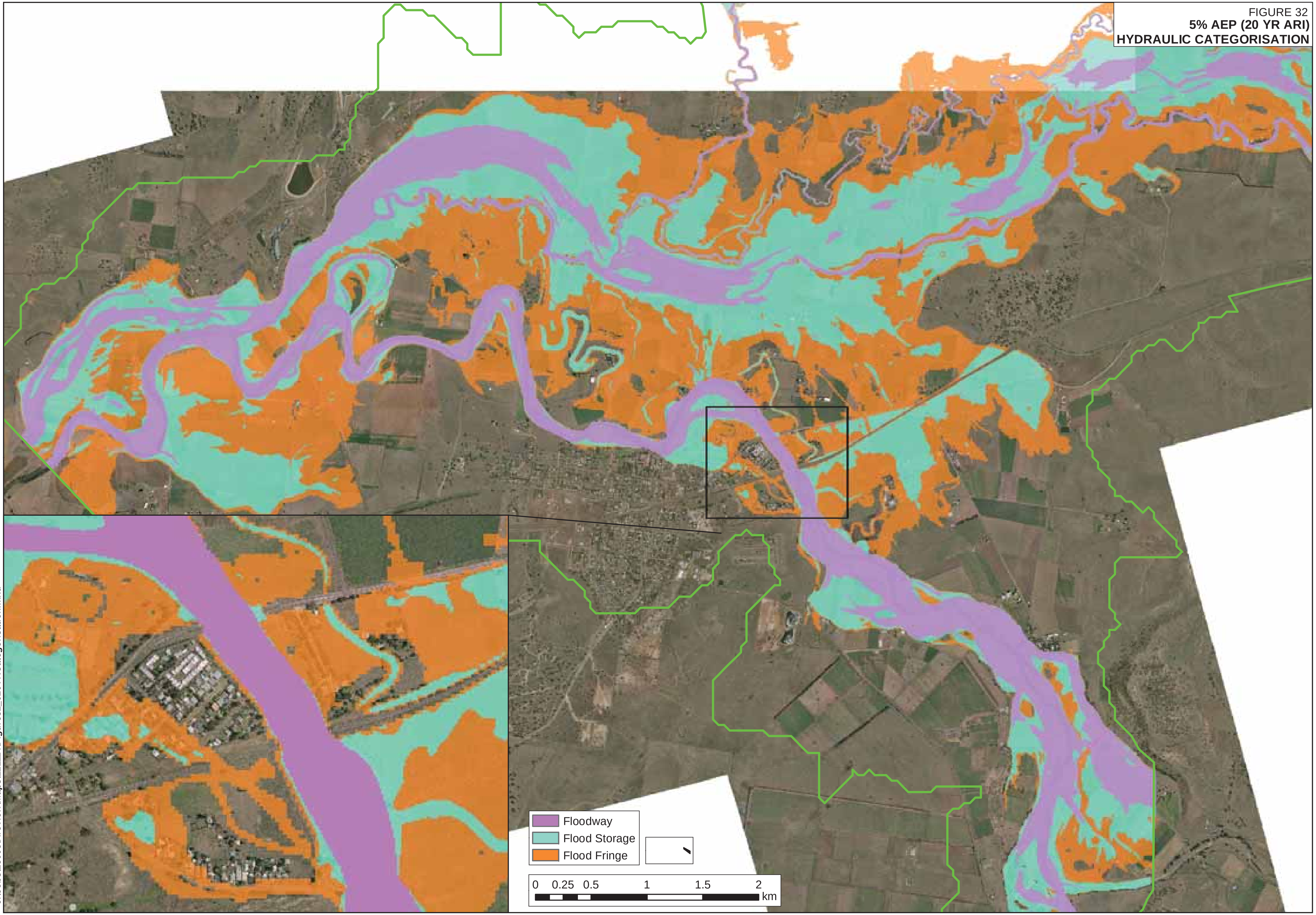

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FIGURE 33
1% AEP (100 YR ARI)
HYDRAULIC CATEGORISATION

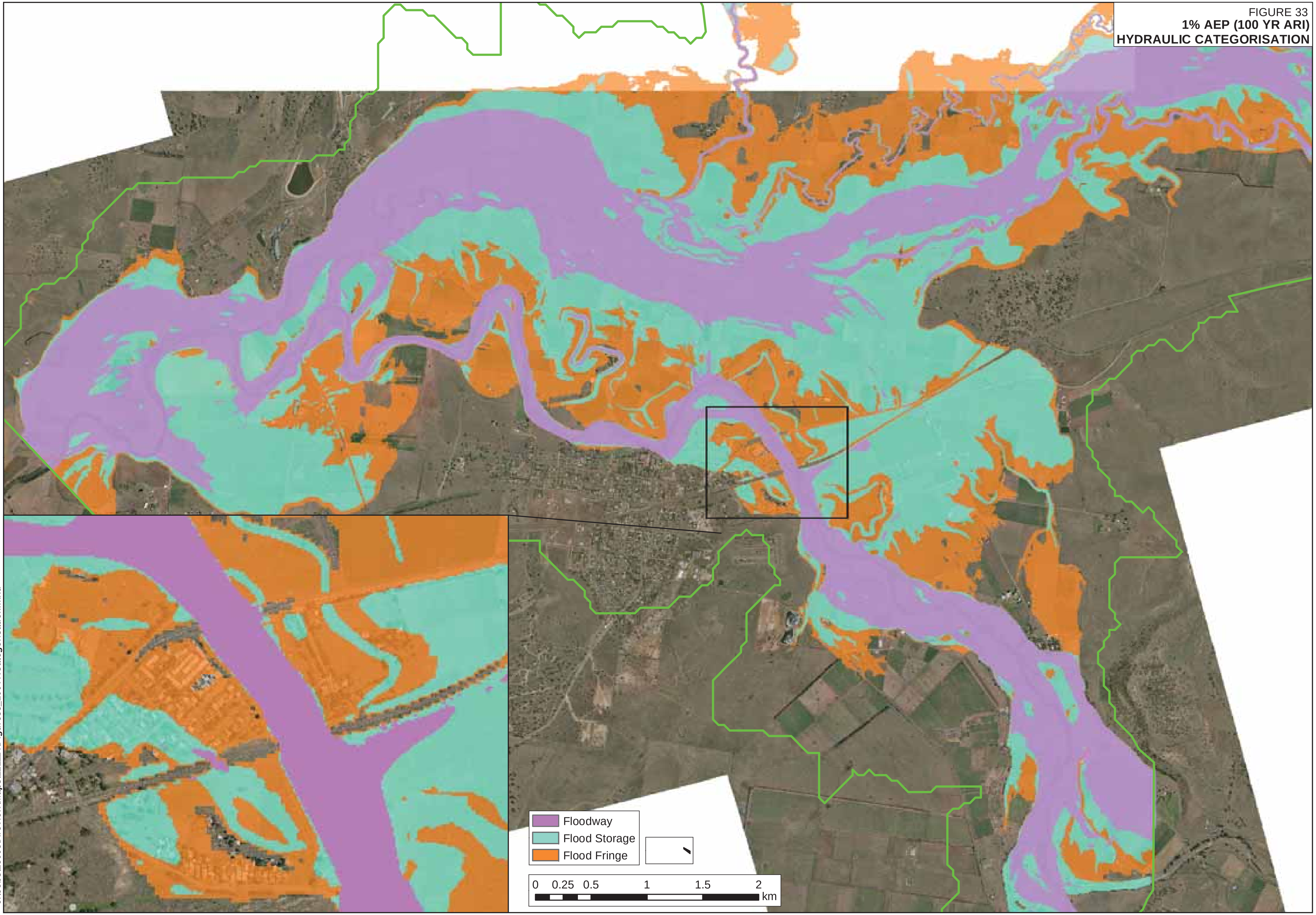

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FIGURE 34
PMF (24 HR GTSMR)
HYDRAULIC CATEGORISATION

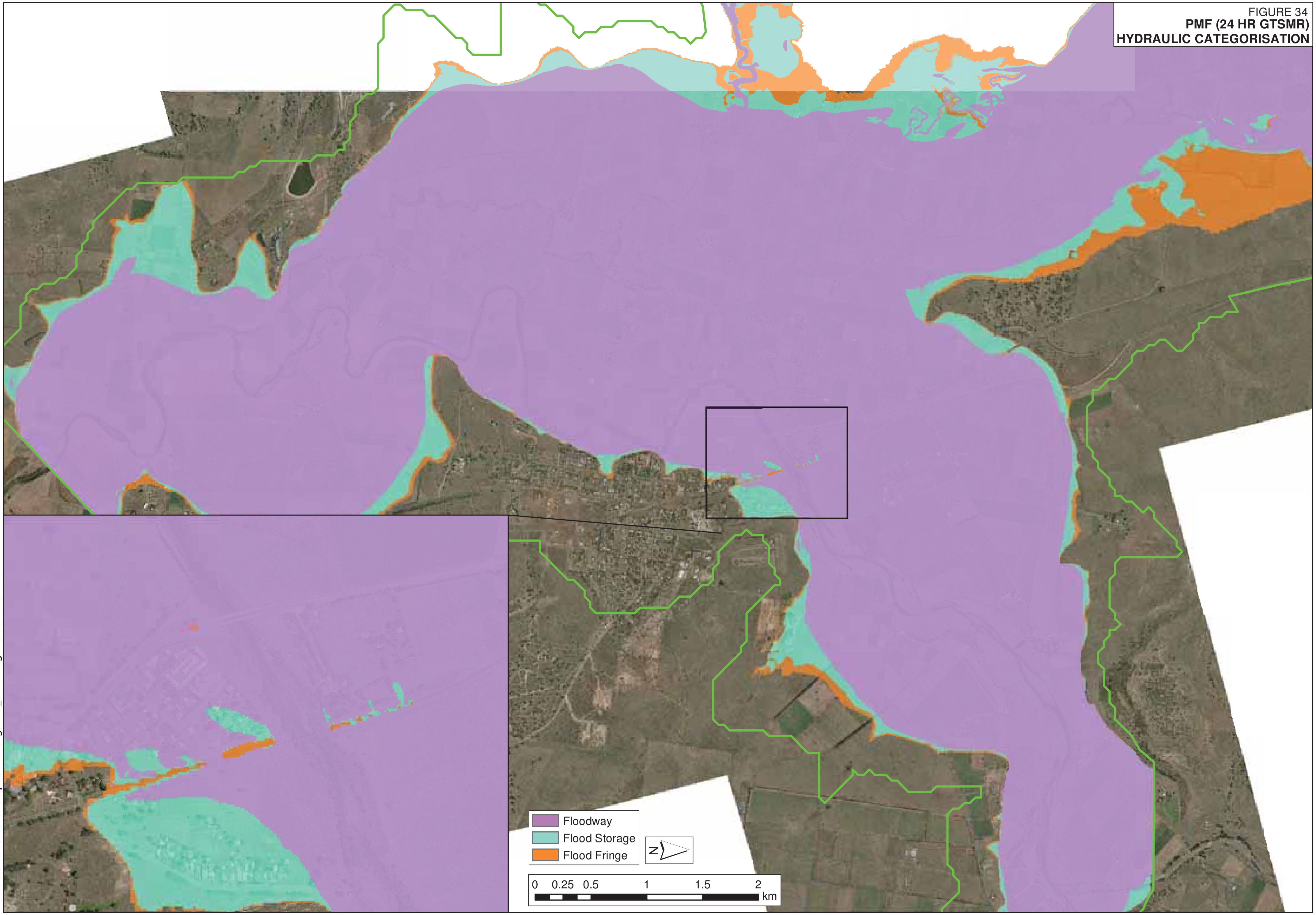

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FIGURE 35
BUILDING FLOOR LEVELS

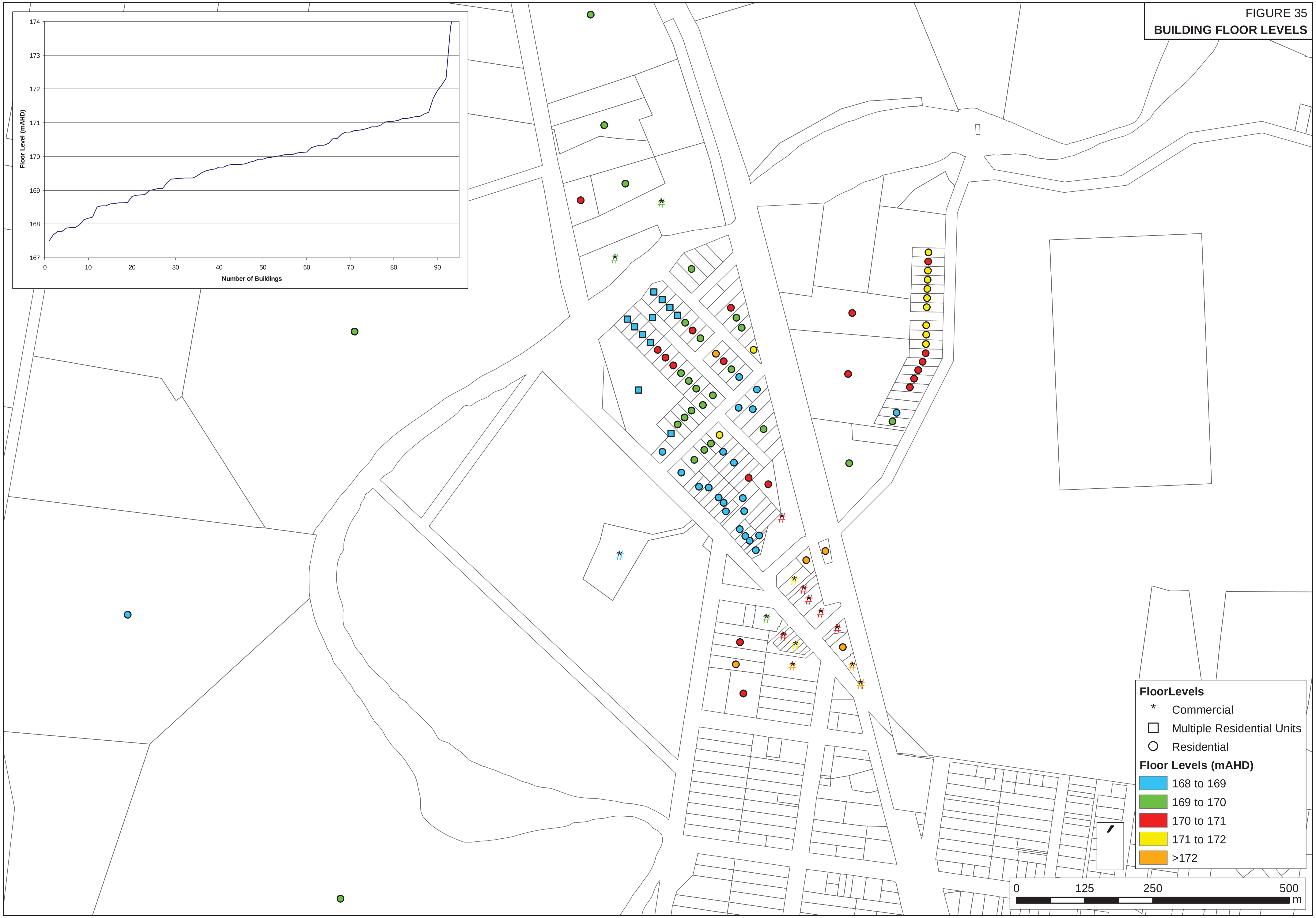

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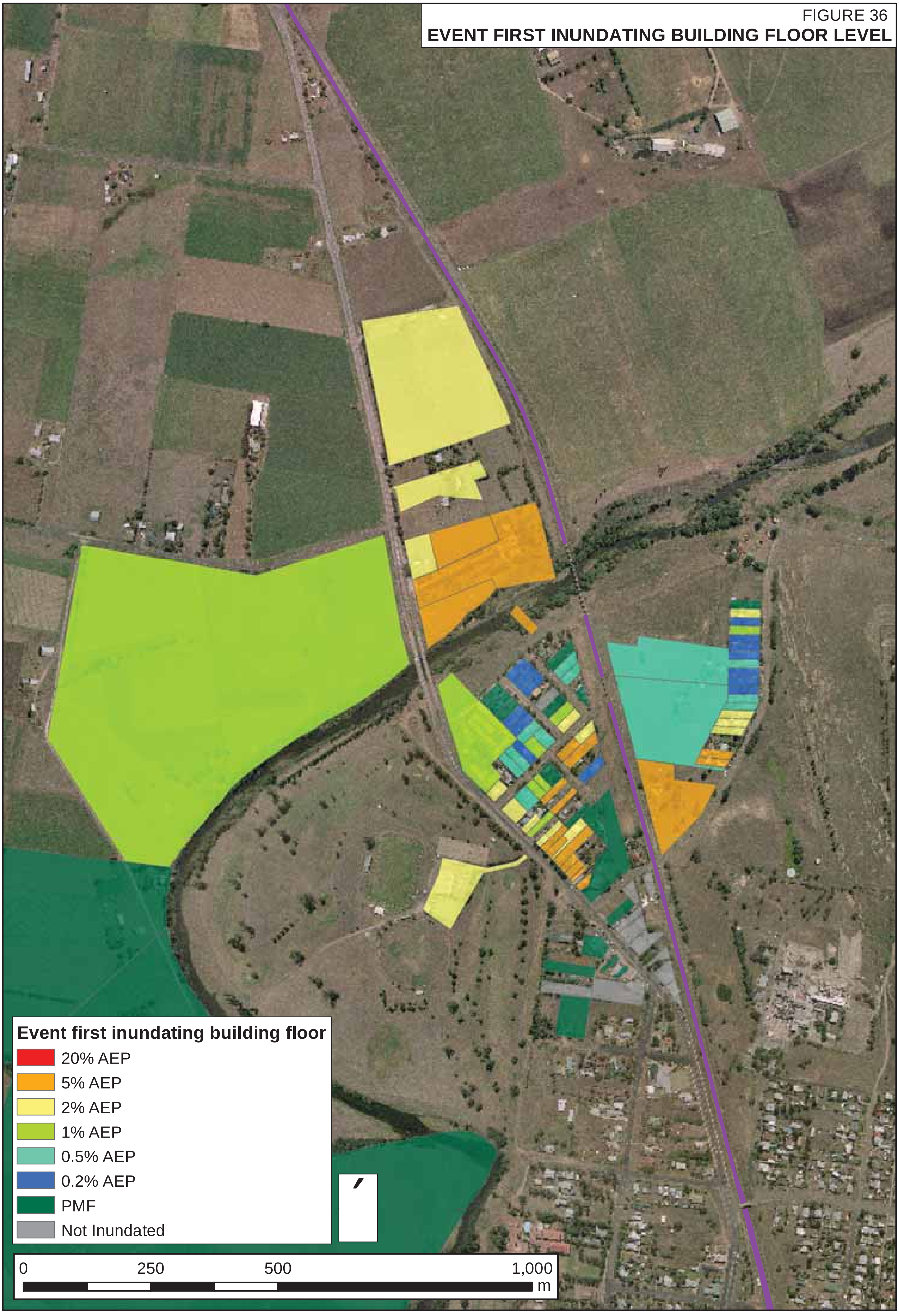
FIGURE 36
EVENT FIRST INUNDATING BUILDING FLOOR LEVEL



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Appendix A

APPENDIX A: GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, Government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power. **redevelopment:** refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated

	response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the

	leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the Aflood liable land@ concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL=s are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the Astandard flood event@ in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of

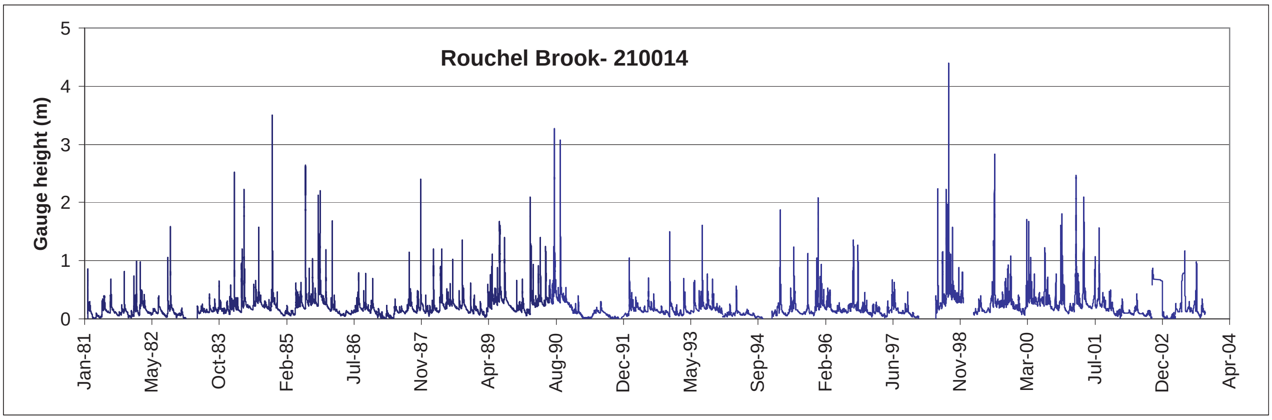
	flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.

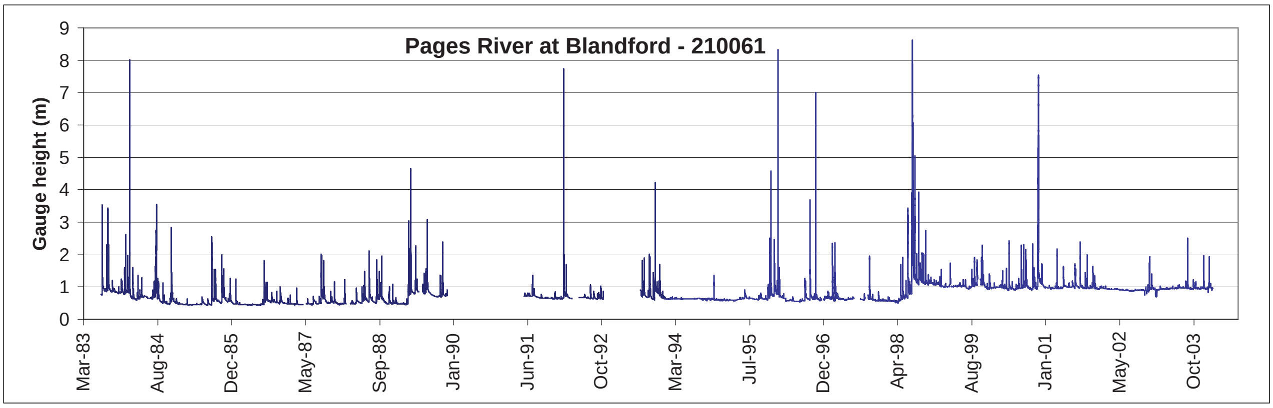
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves:
	$ the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or
	$ water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or
	$ major overland flow paths through developed areas outside of defined drainage reserves; and/or
	$ the potential to affect a number of buildings along the major flow path.

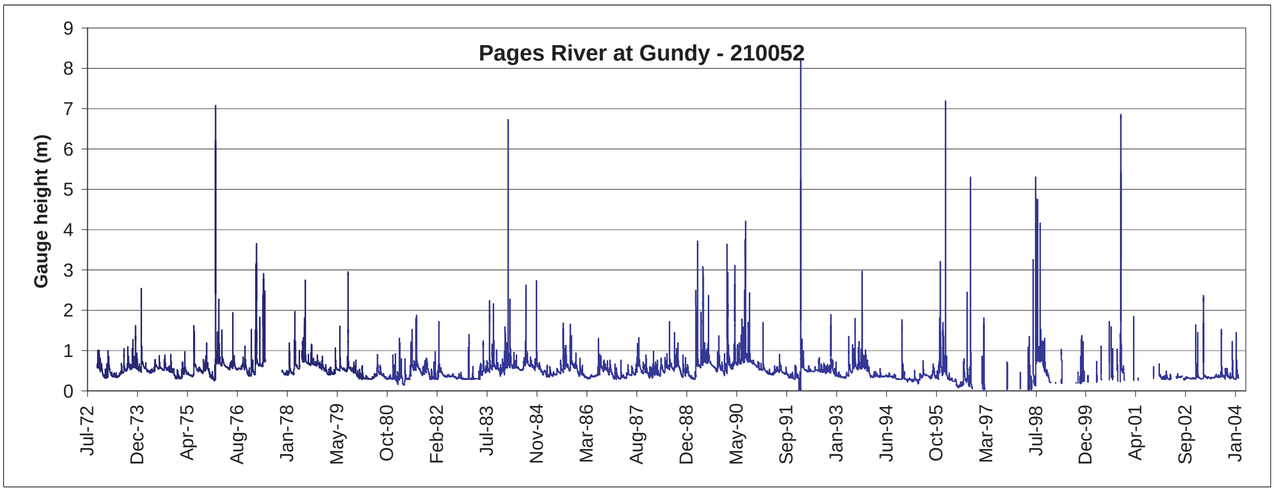
mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State=s rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.

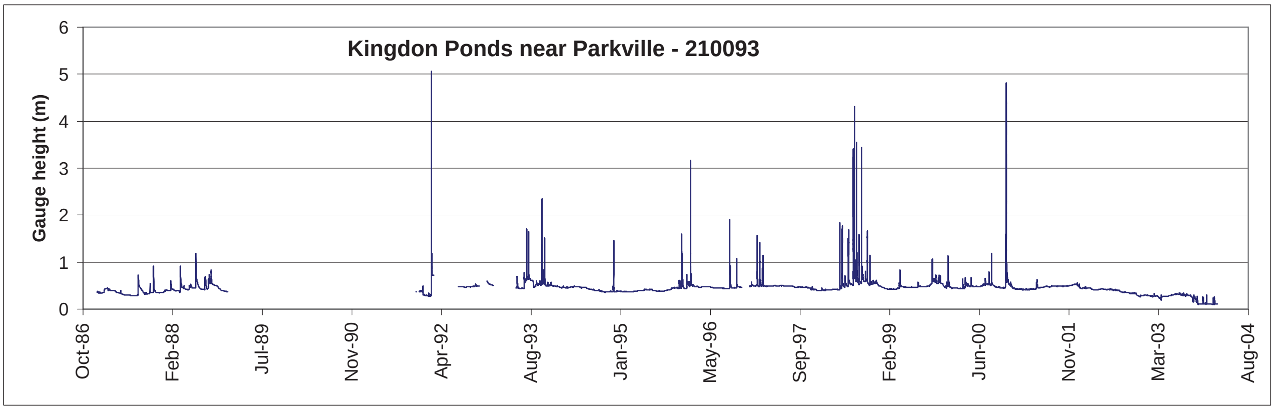
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to Awater level@. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

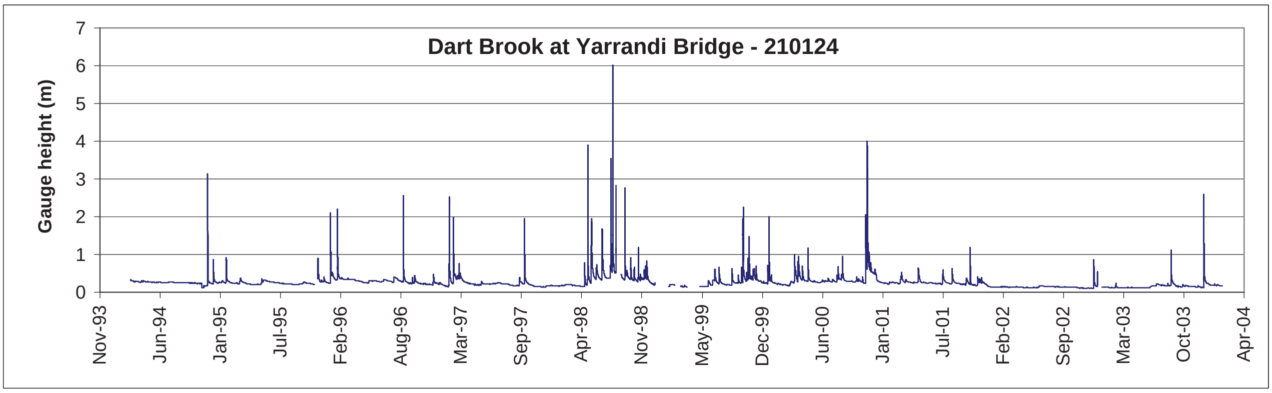
Appendix B

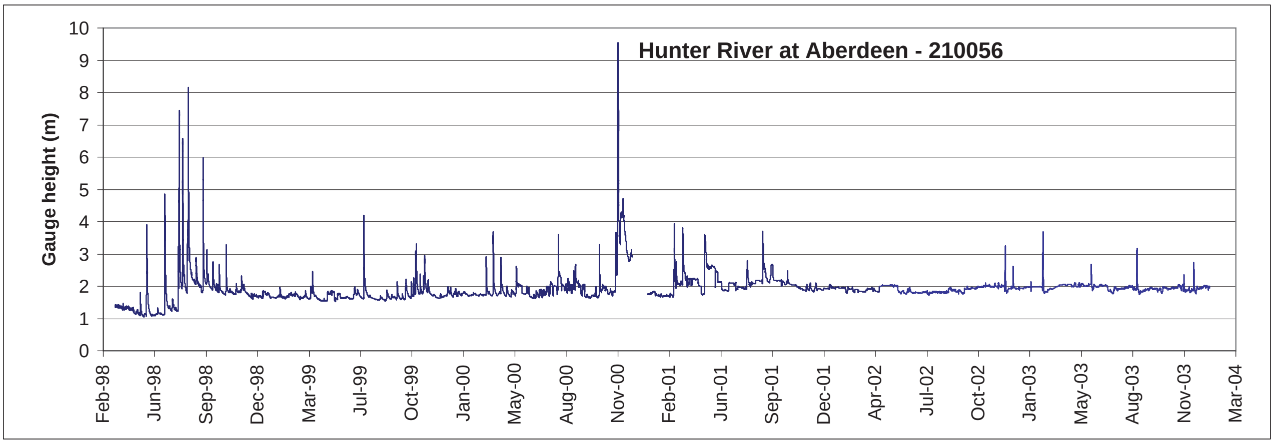

Rouchel Brook- 210014
Gauge height (m)
0
1
2
3
4
5
Jan-81
May-82
Oct-83
Feb-85
Jul-86
Nov-87
Apr-89
Aug-90
Dec-91
May-93
Sep-94
Feb-96
Jun-97
Nov-98
Mar-00
Jul-01
Dec-02
Apr-04

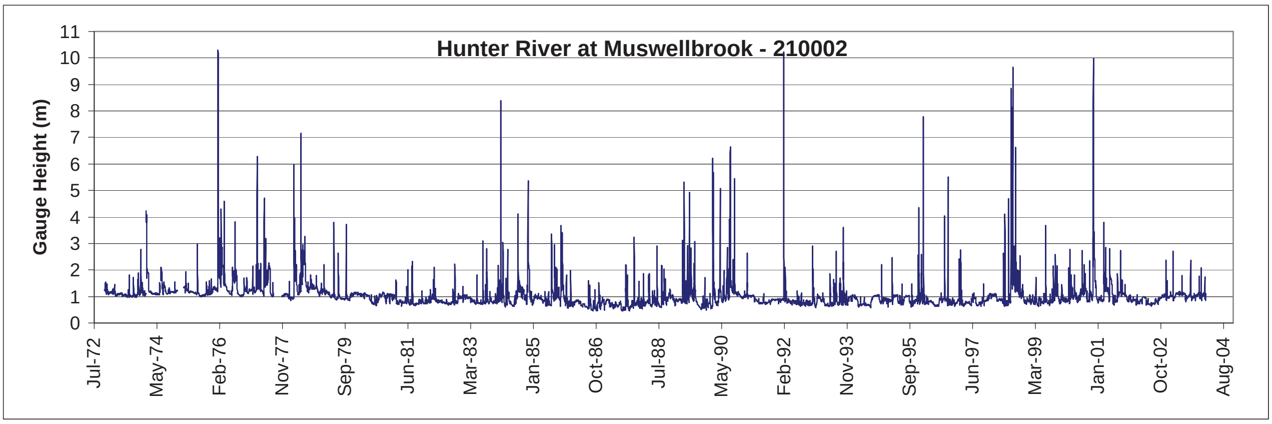

Pages River at Blandford - 210061
Gauge height (m)
0
1
2
3
4
5
6
7
8
9
Mar-83
Aug-84
Dec-85
May-87
Sep-88
Jan-90
Jun-91
Oct-92
Mar-94
Jul-95
Dec-96
Apr-98
Aug-99
Jan-01
May-02
Oct-03


Pages River at Gundy - 210052
Gauge height (m)
0
1
2
3
4
5
6
7
8
9
Jul-72
Dec-73
Apr-75
Aug-76
Jan-78
May-79
Oct-80
Feb-82
Jul-83
Nov-84
Mar-86
Aug-87
Dec-88
May-90
Sep-91
Jan-93
Jun-94
Oct-95
Mar-97
Jul-98
Dec-99
Apr-01
Sep-02
Jan-04


Kingdon Ponds near Parkville - 210093
Gauge height (m)
0
1
2
3
4
5
6
Oct-86
Feb-88
Jul-89
Nov-90
Apr-92
Aug-93
Jan-95
May-96
Sep-97
Feb-99
Jun-00
Nov-01
Mar-03
Aug-04


Dart Brook at Yarrandi Bridge - 210124
Gauge height (m)
0
1
2
3
4
5
6
7
Nov-93
Jun-94
Jan-95
Jul-95
Feb-96
Aug-96
Mar-97
Sep-97
Apr-98
Nov-98
May-99
Dec-99
Jun-00
Jan-01
Jul-01
Feb-02
Sep-02
Mar-03
Oct-03
Apr-04


Hunter River at Aberdeen - 210056
Gauge height (m)
0
1
2
3
4
5
6
7
8
9
10
Feb-98
Jun-98
Sep-98
Dec-98
Mar-99
Jul-99
Oct-99
Jan-00
May-00
Aug-00
Nov-00
Feb-01
Jun-01
Sep-01
Dec-01
Apr-02
Jul-02
Oct-02
Jan-03
May-03
Aug-03
Nov-03
Mar-04


Hunter River at Muswellbrook - 210002
Gauge Height (m)
0
1
2
3
4
5
6
7
8
9
10
11
Jul-72
May-74
Feb-76
Nov-77
Sep-79
Jun-81
Mar-83
Jan-85
Oct-86
Jul-88
May-90
Feb-92
Nov-93
Sep-95
Jun-97
Mar-99
Jan-01
Oct-02
Aug-04

Appendix C

FIGURE C1

CUMULATIVE RAINFALL JANUARY 1984

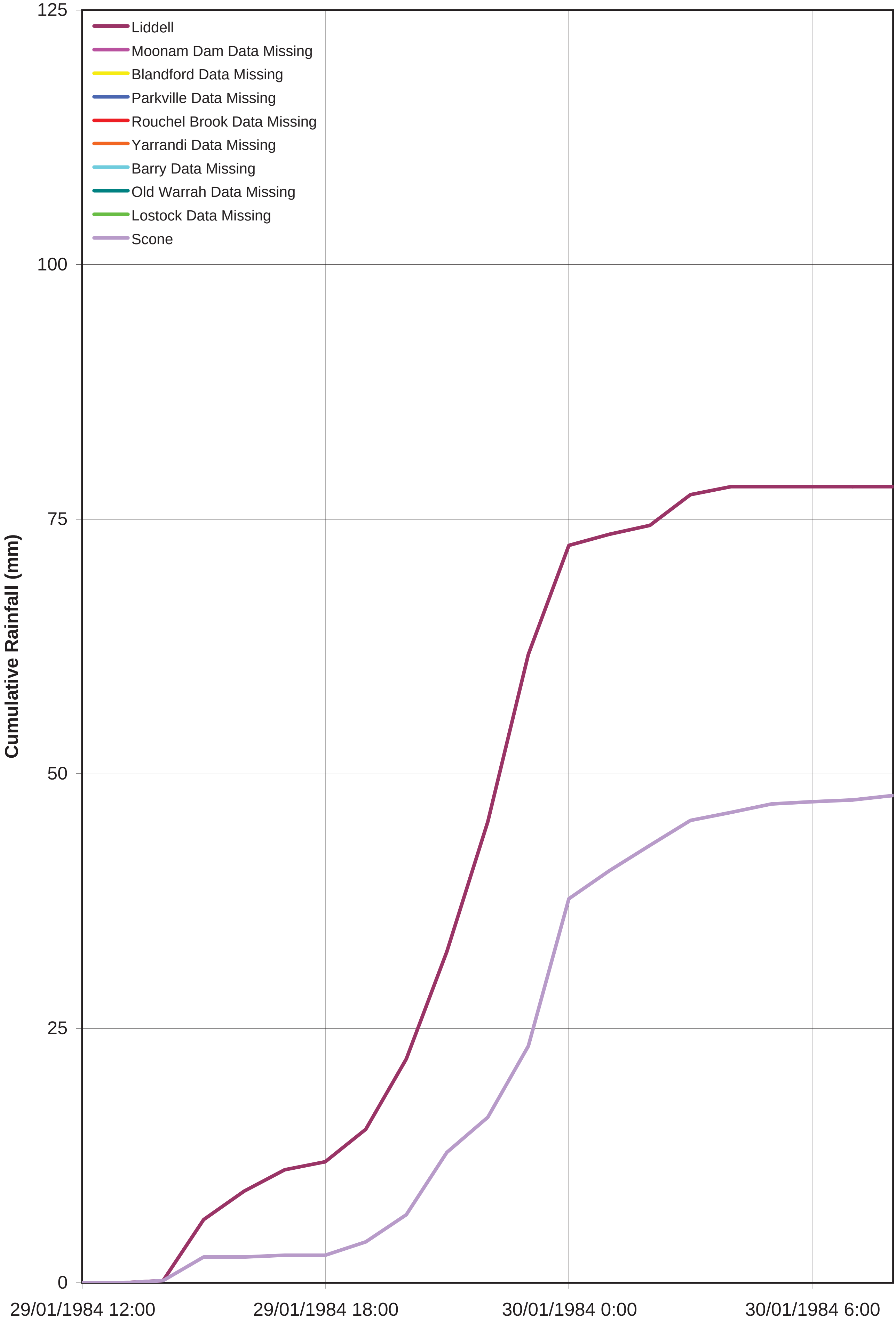


FIGURE C2

CUMULATIVE RAINFALL FEBRUARY 1992

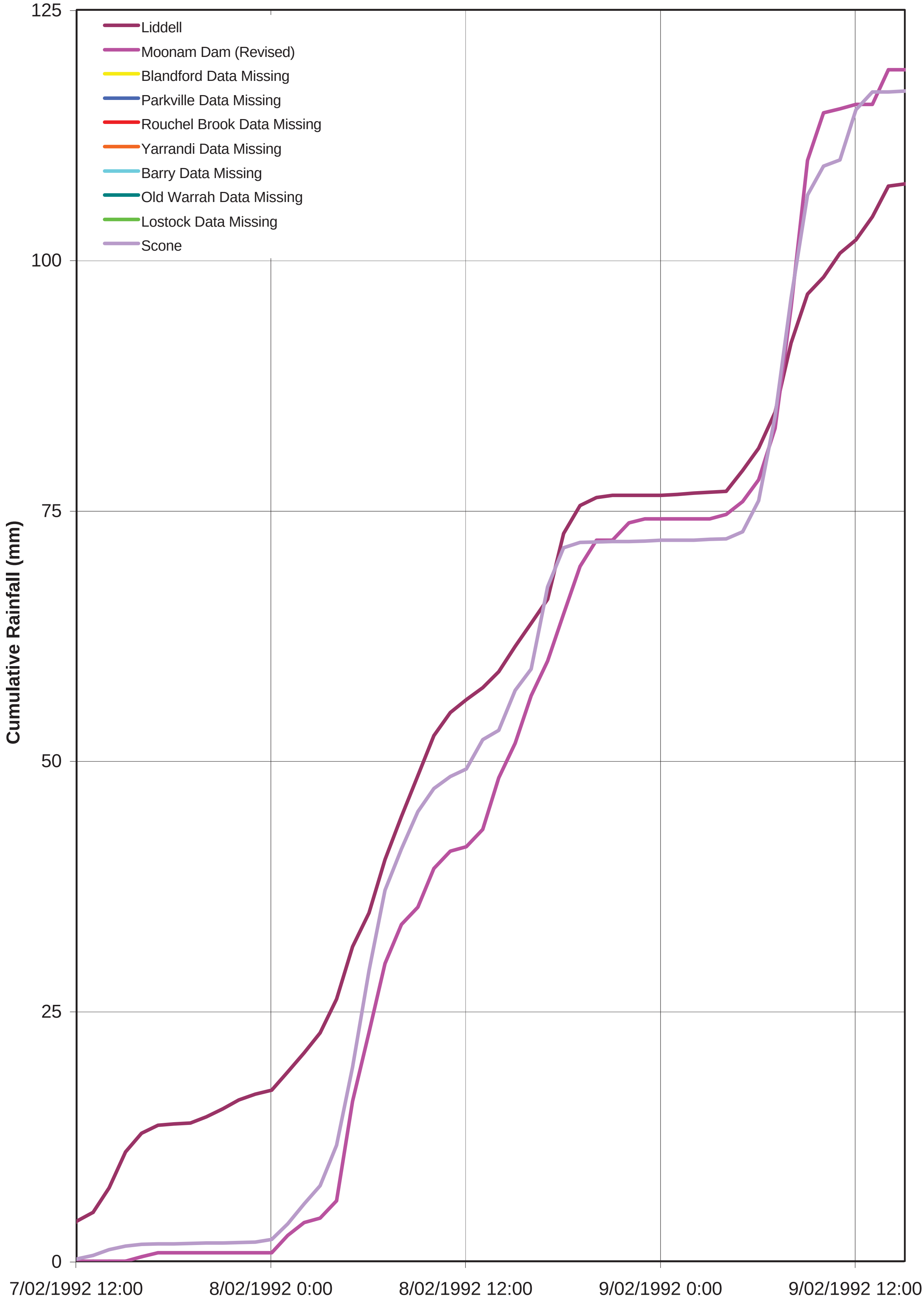


FIGURE C3

CUMULATIVE RAINFALL JANUARY 1996

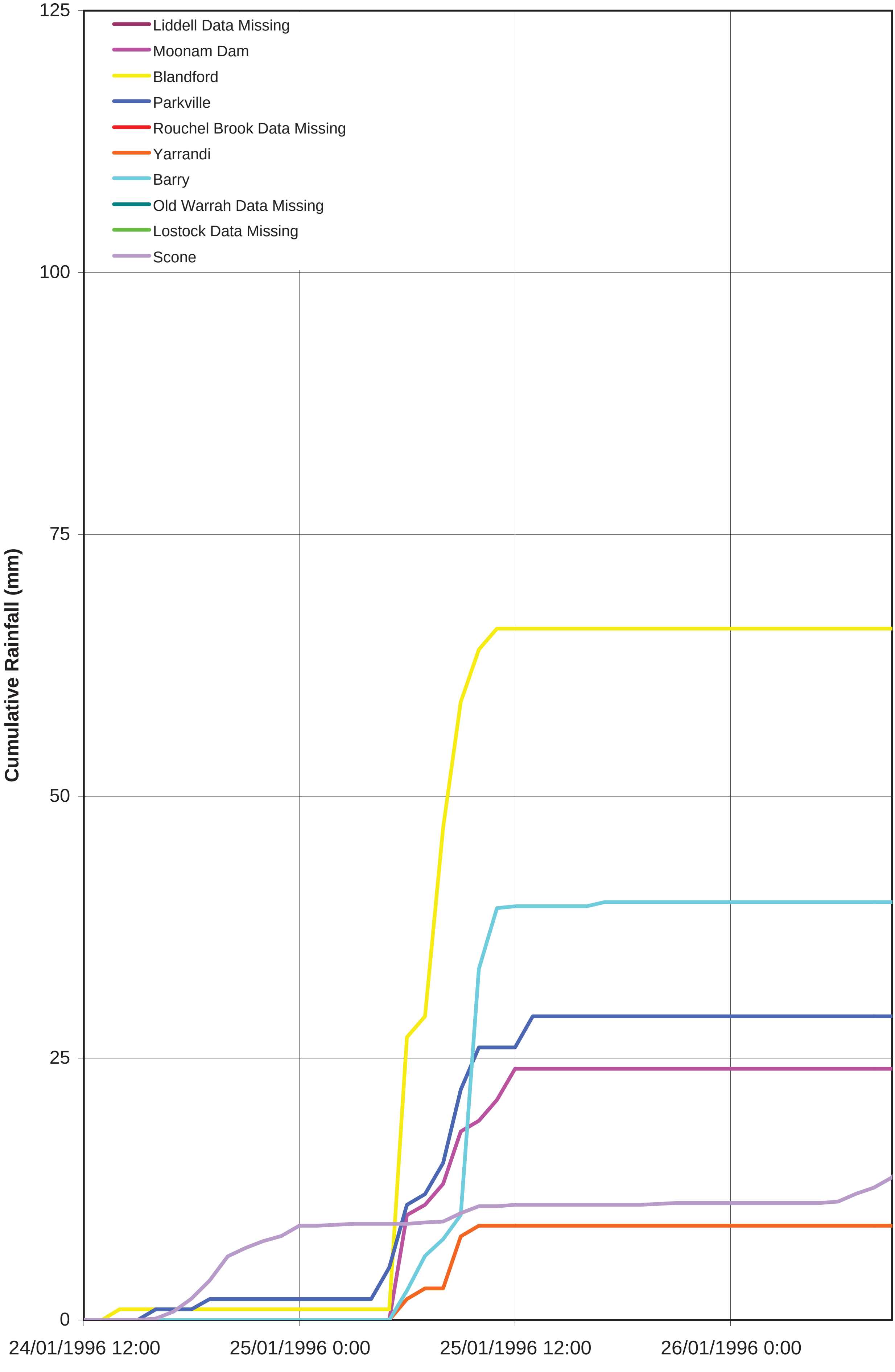


FIGURE C4

CUMULATIVE RAINFALL JULY 21st 1998

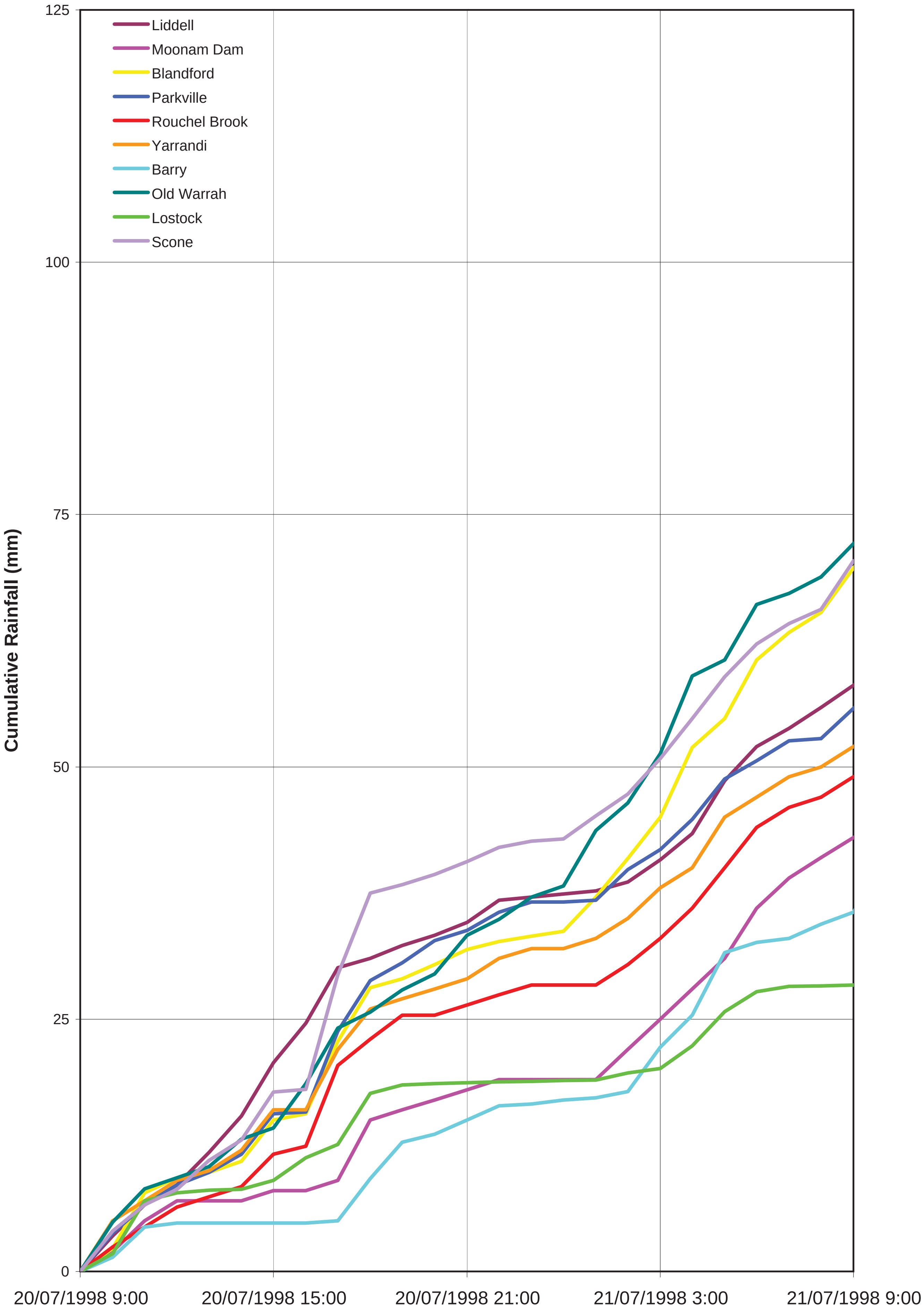


FIGURE C5

CUMULATIVE RAINFALL JULY 28th 1998

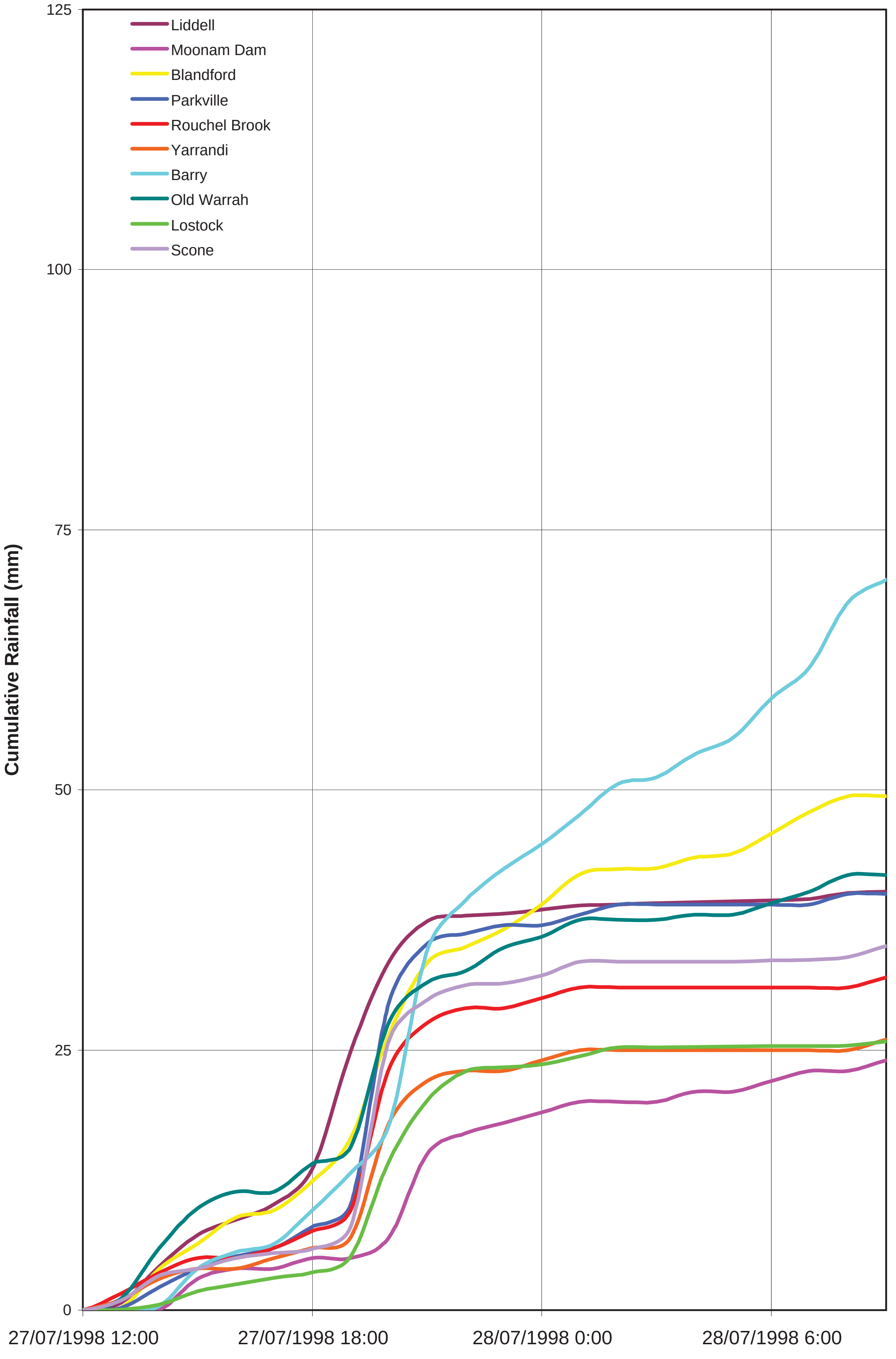


FIGURE C6

CUMULATIVE RAINFALL AUGUST 1998

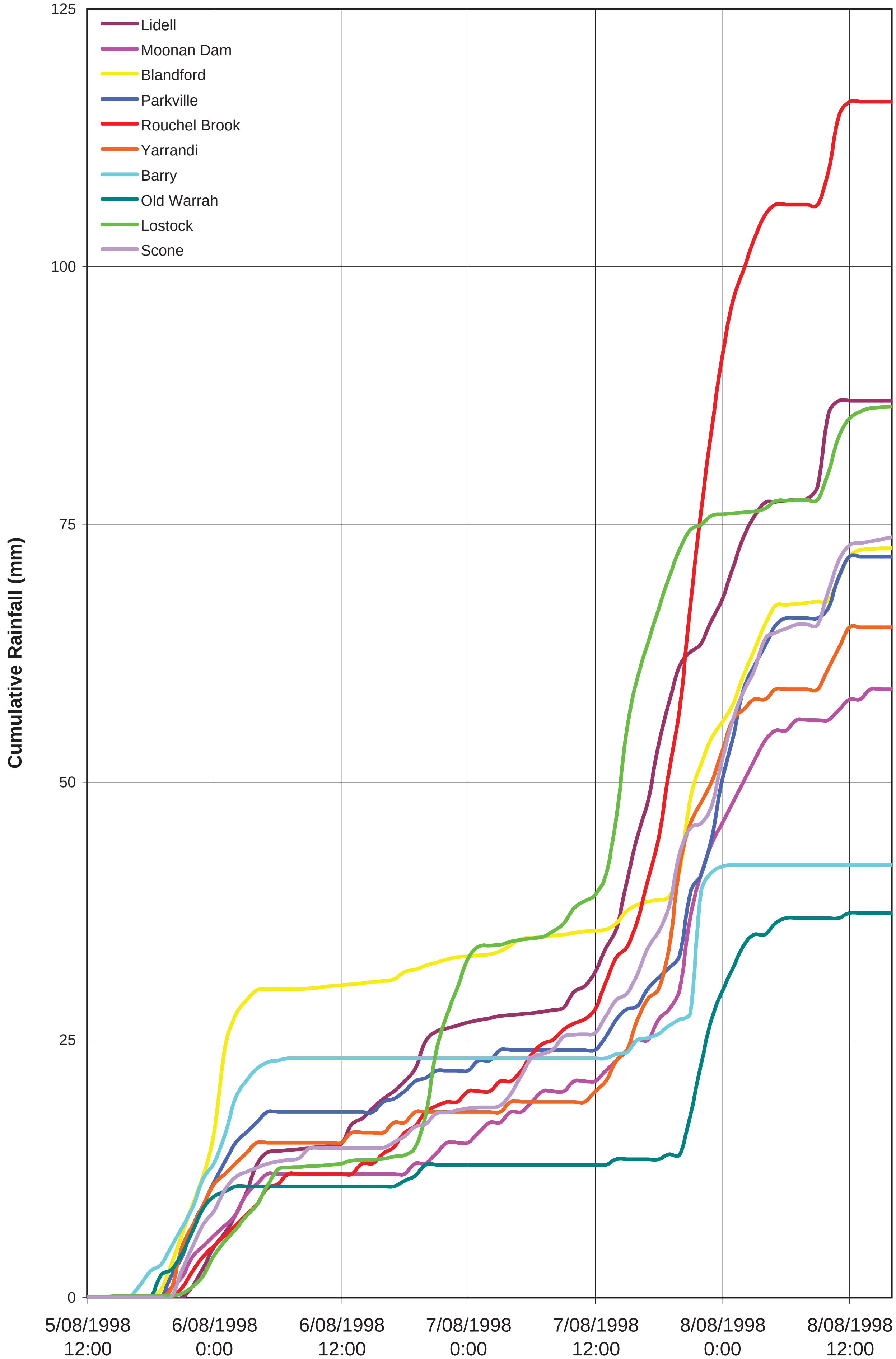


FIGURE C7

CUMULATIVE RAINFALL NOVEMBER 2000

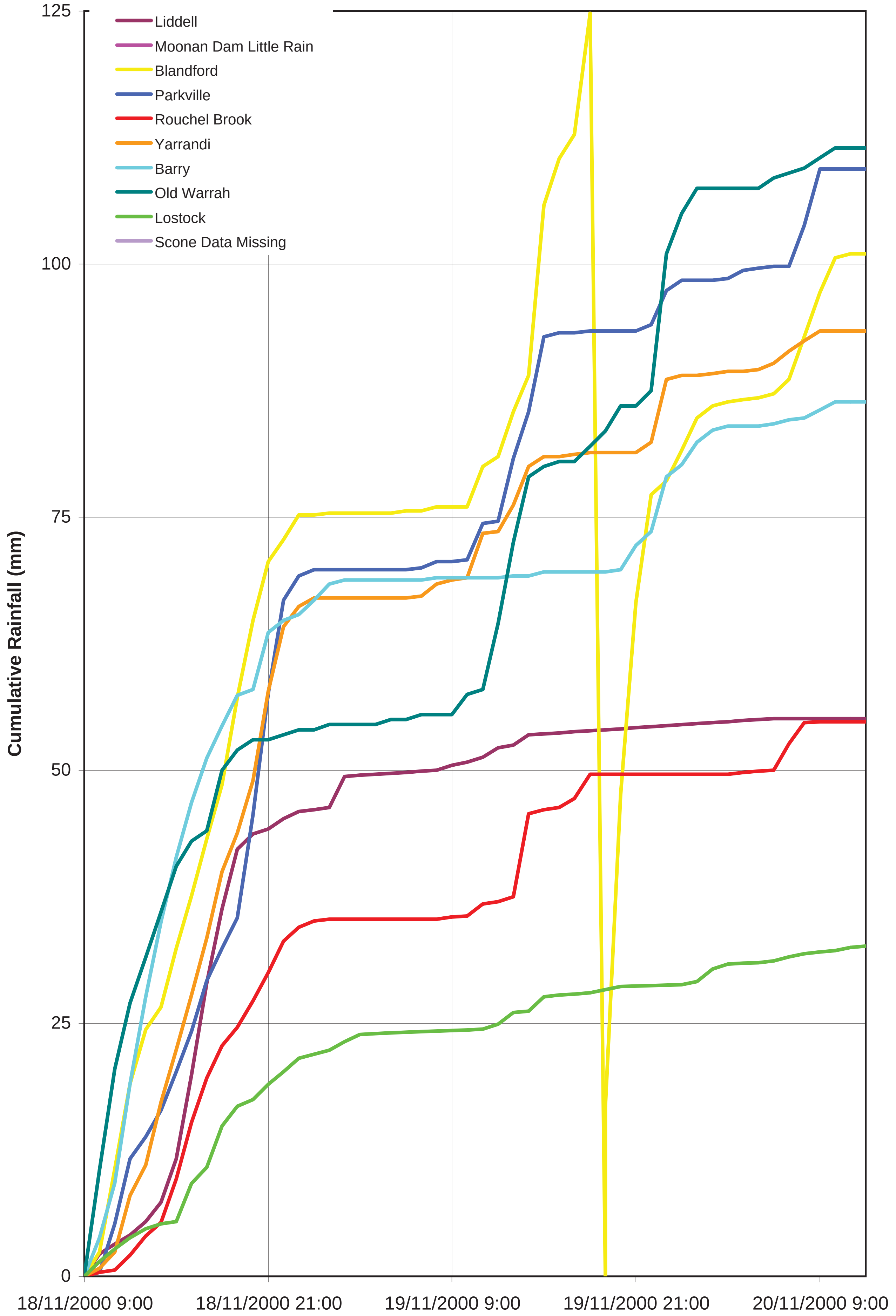


FIGURE C8
FLOW HYDROGRAPHS
JANUARY 1984

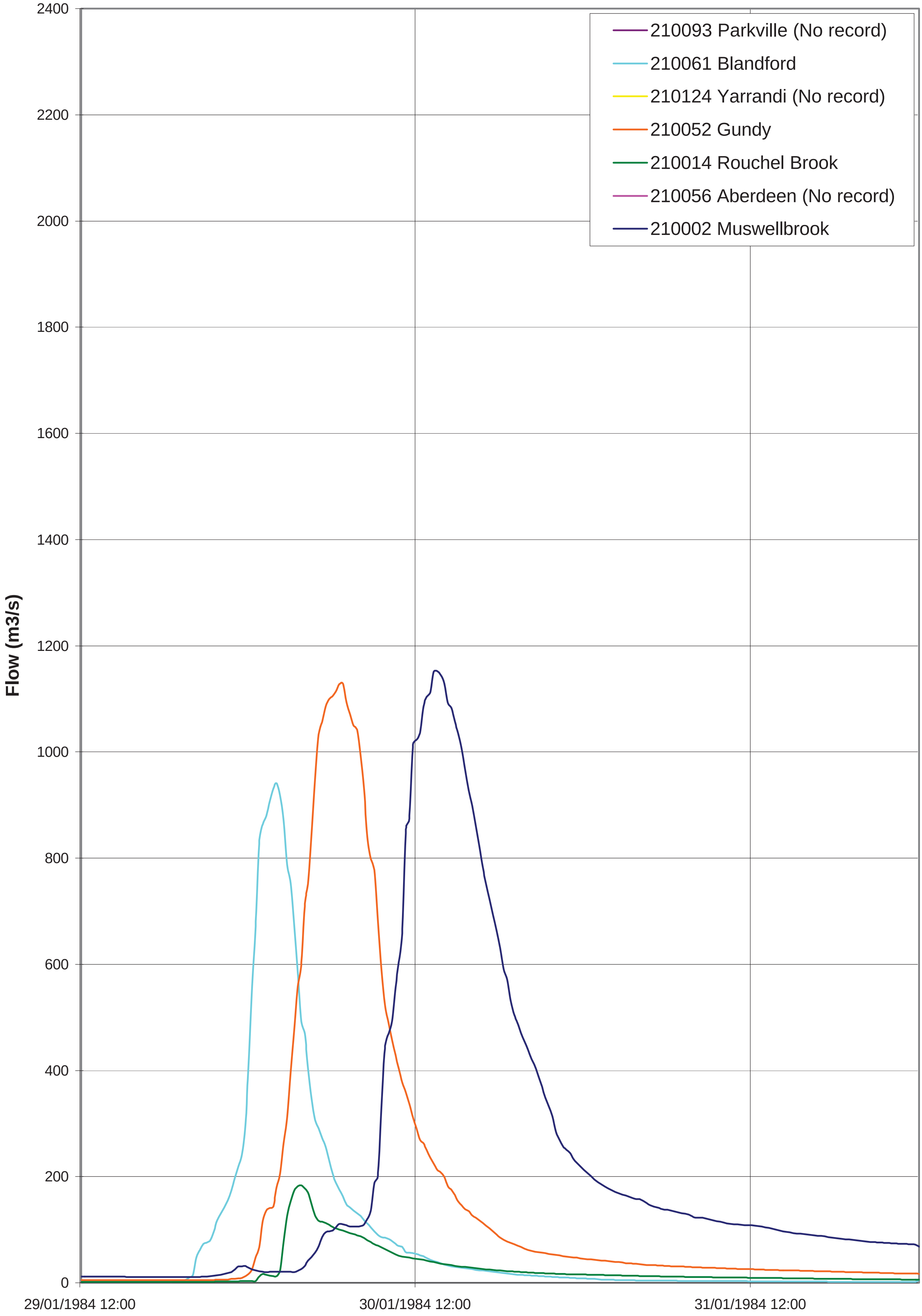


FIGURE C9

FLOW HYDROGRAPHS

FEBRUARY 1992

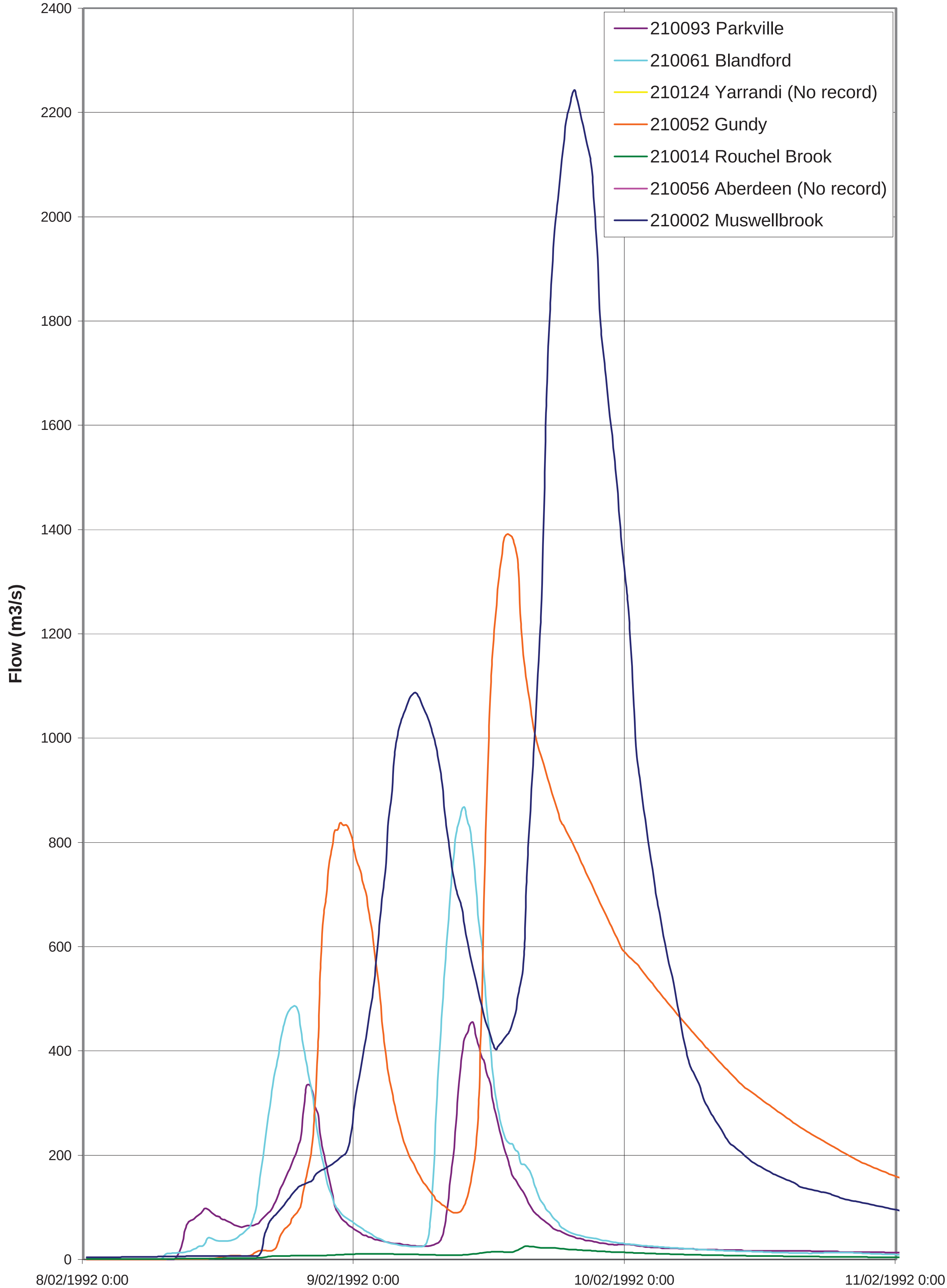


FIGURE C10
FLOW HYDROGRAPHS
JANUARY 1996

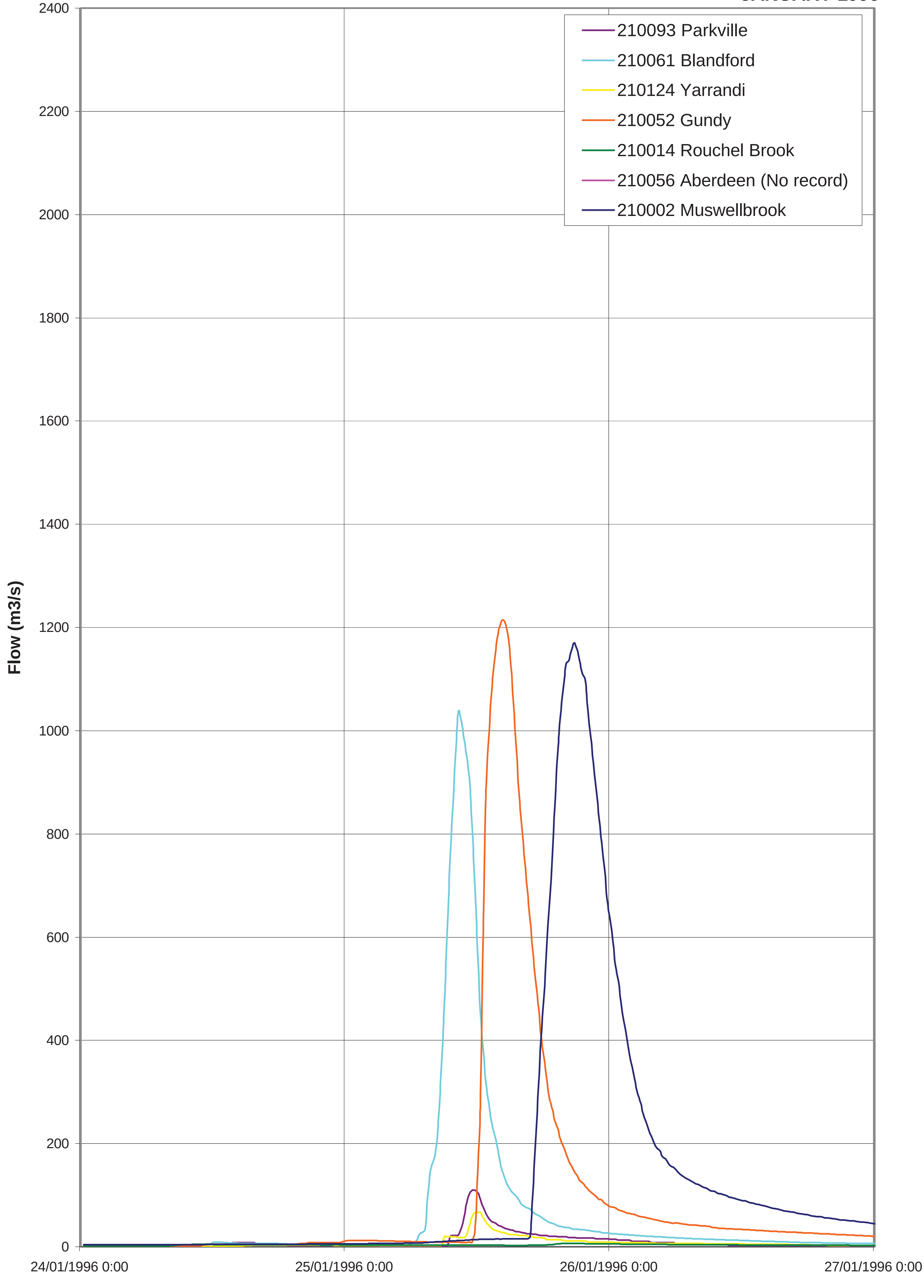


FIGURE C11

FLOW HYDROGRAPHS

JULY 21ST 1998

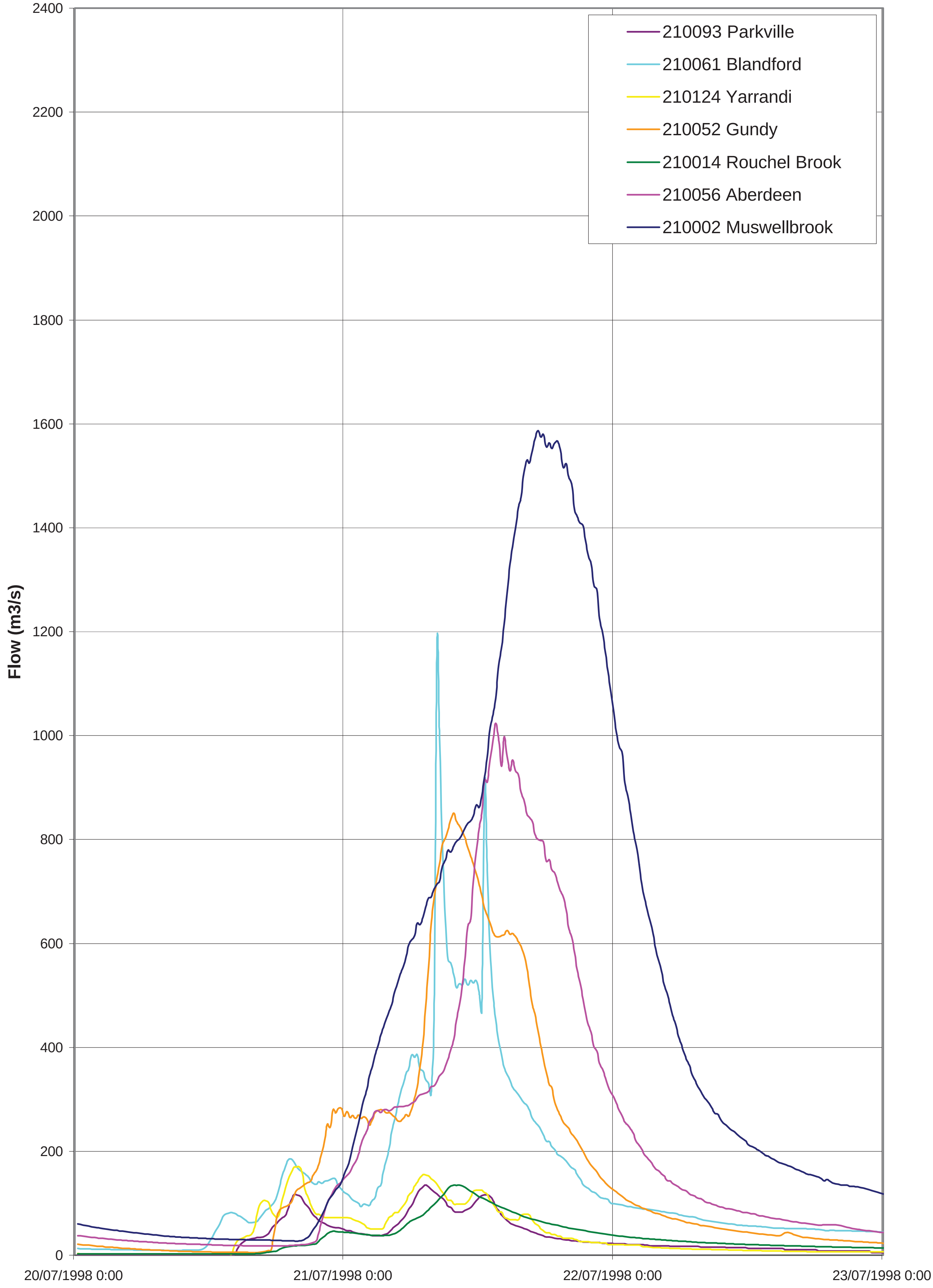


FIGURE C12

FLOW HYDROGRAPHS

JULY 28TH 1998

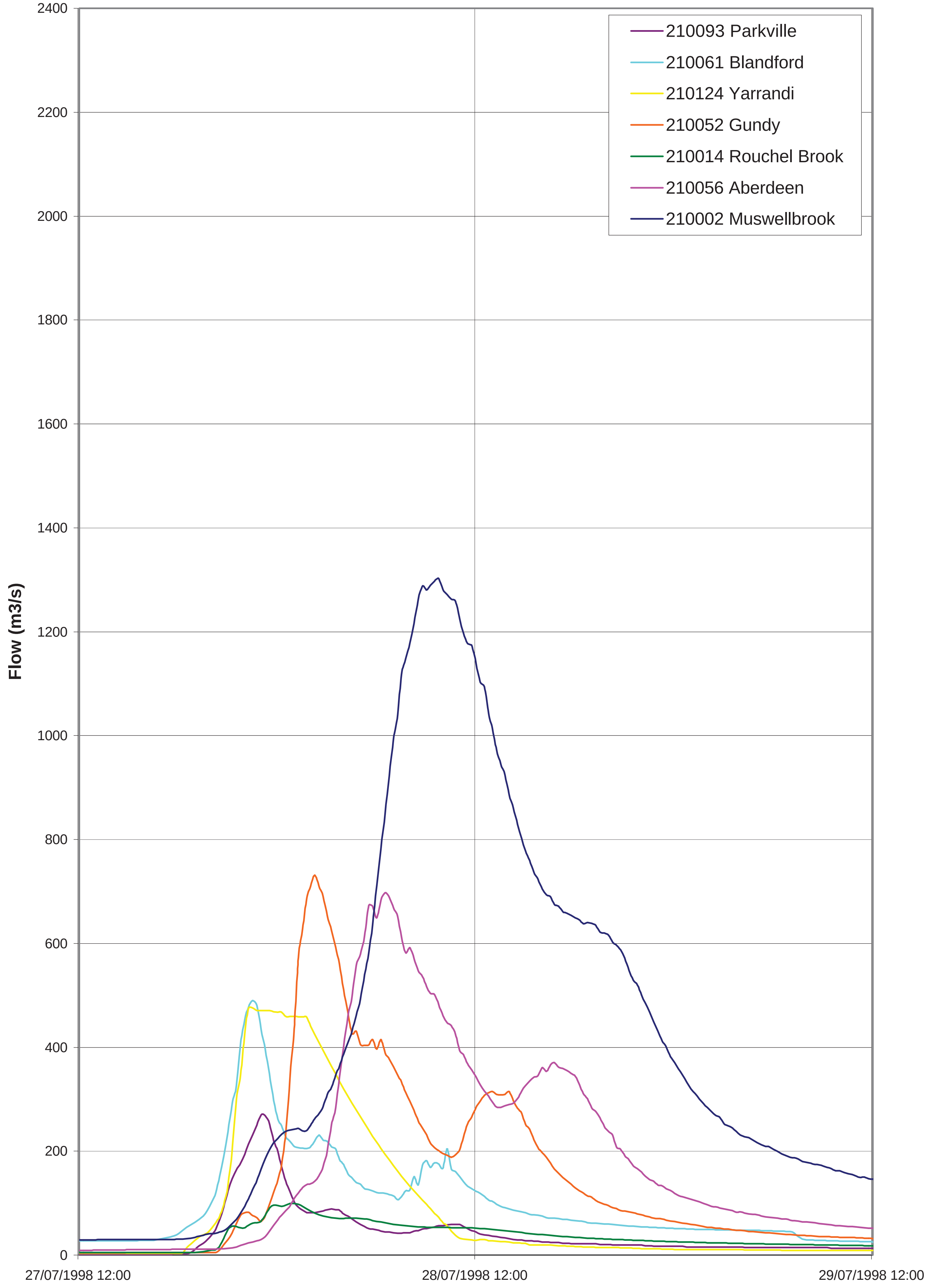


FIGURE C13

FLOW HYDROGRAPHS
AUGUST 1998

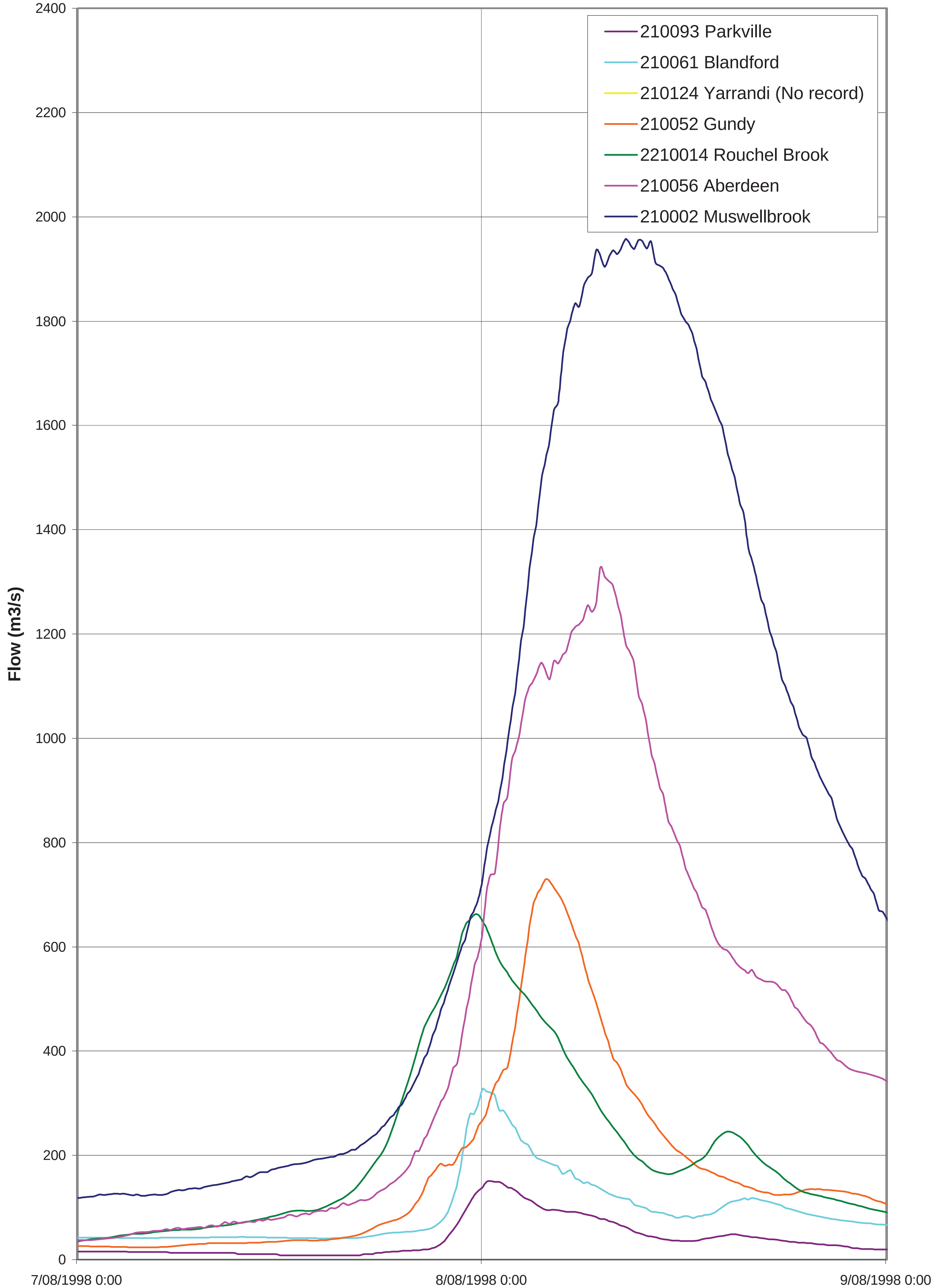


FIGURE C14

FLOW HYDROGRAPHS
NOVEMBER 2000

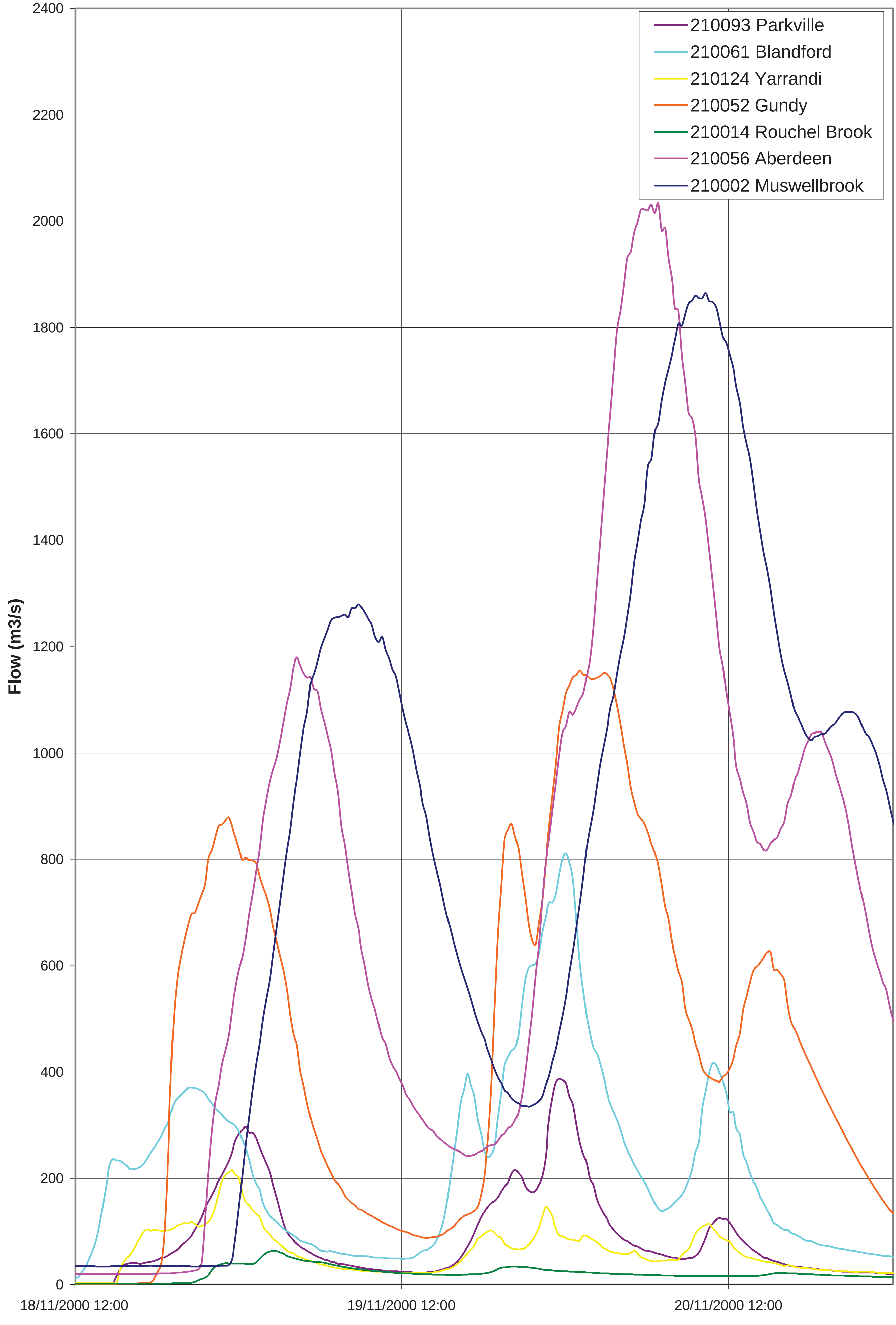
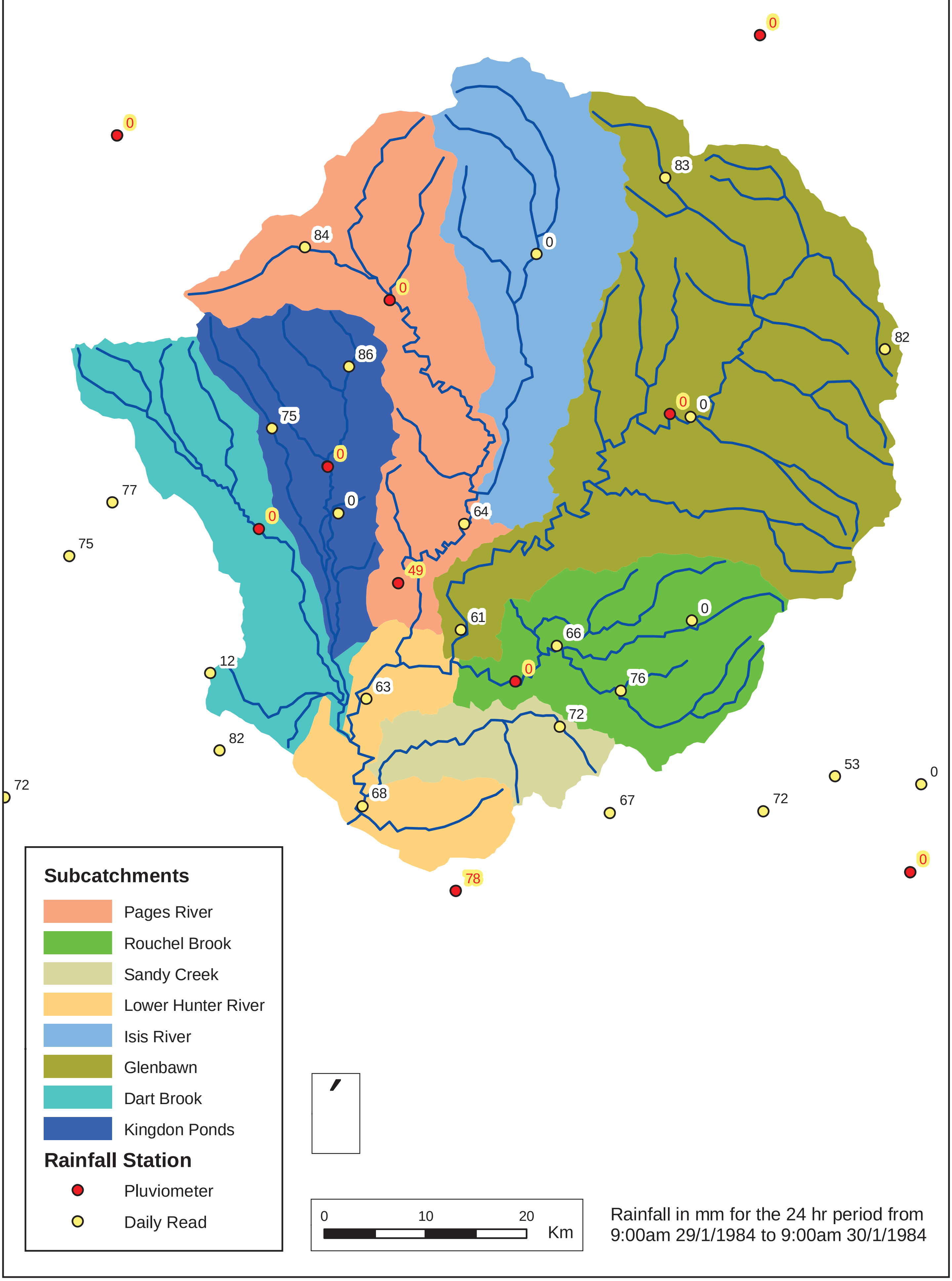


FIGURE C15

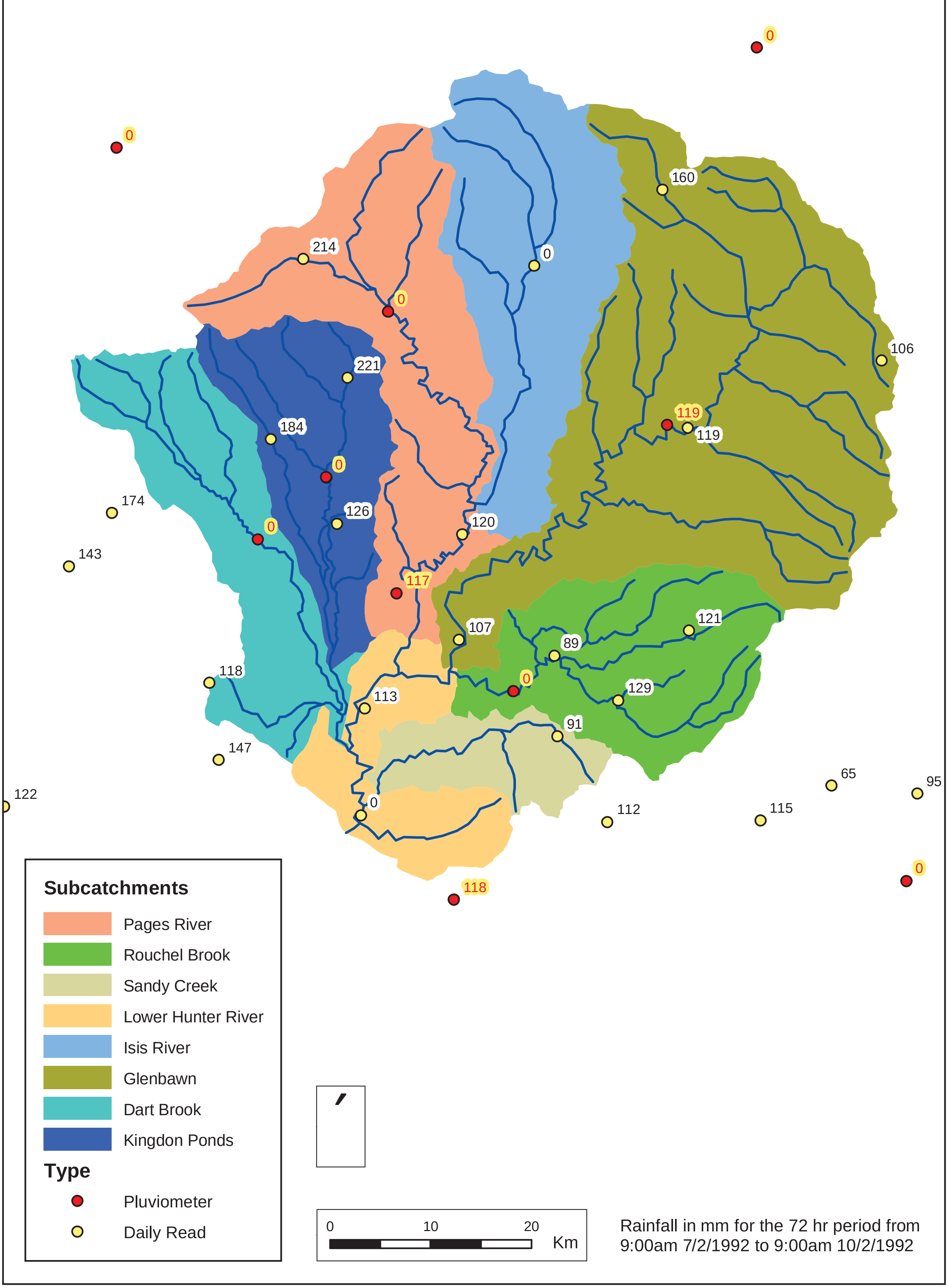
RAINFALL JANUARY 1984



Rainfall in mm for the 24 hr period from 9:00am 29/1/1984 to 9:00am 30/1/1984

FIGURE C16

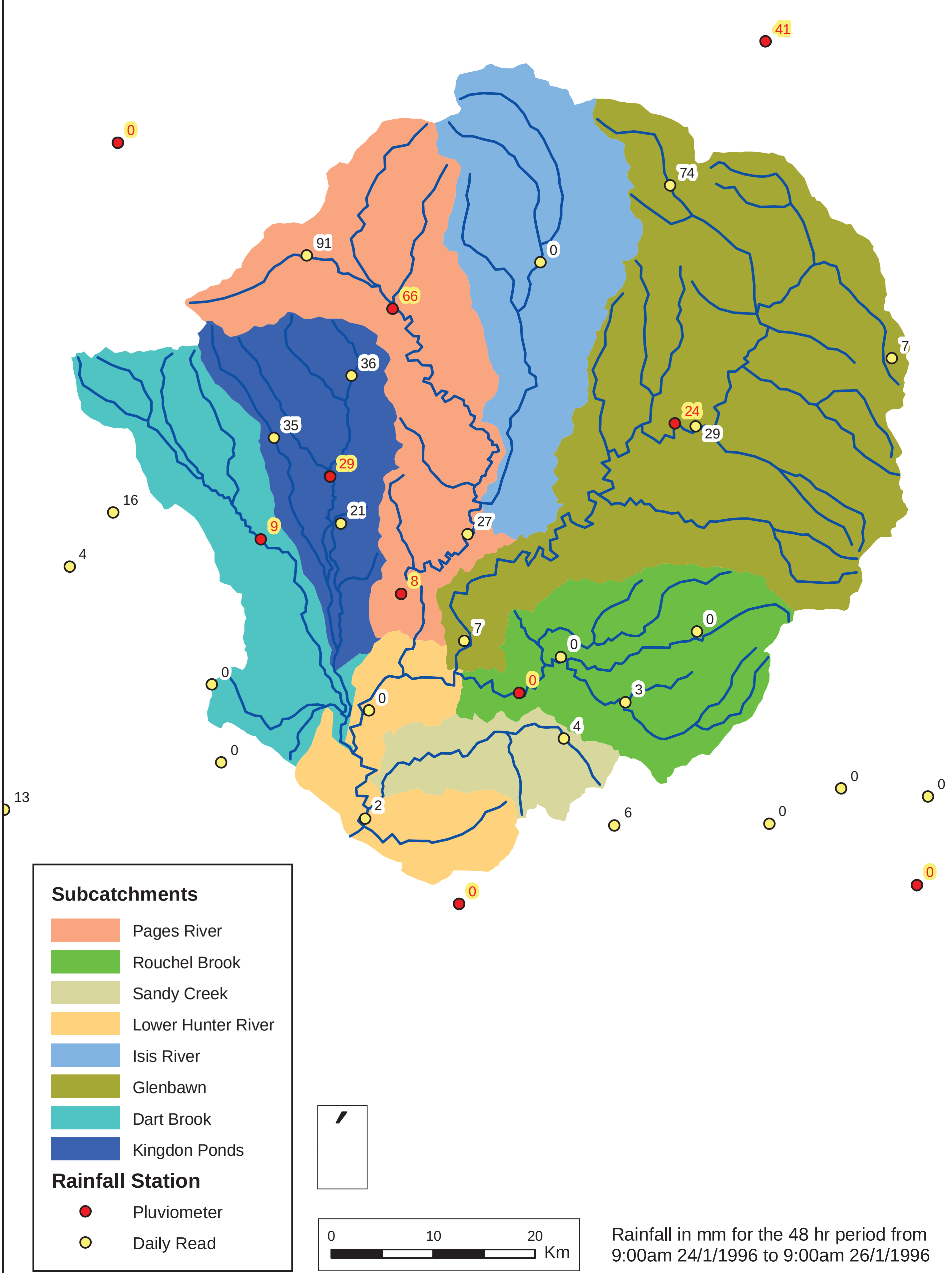
RAINFALL FEBRUARY 1992



Rainfall in mm for the 72 hr period from 9:00am 7/2/1992 to 9:00am 10/2/1992

FIGURE C17

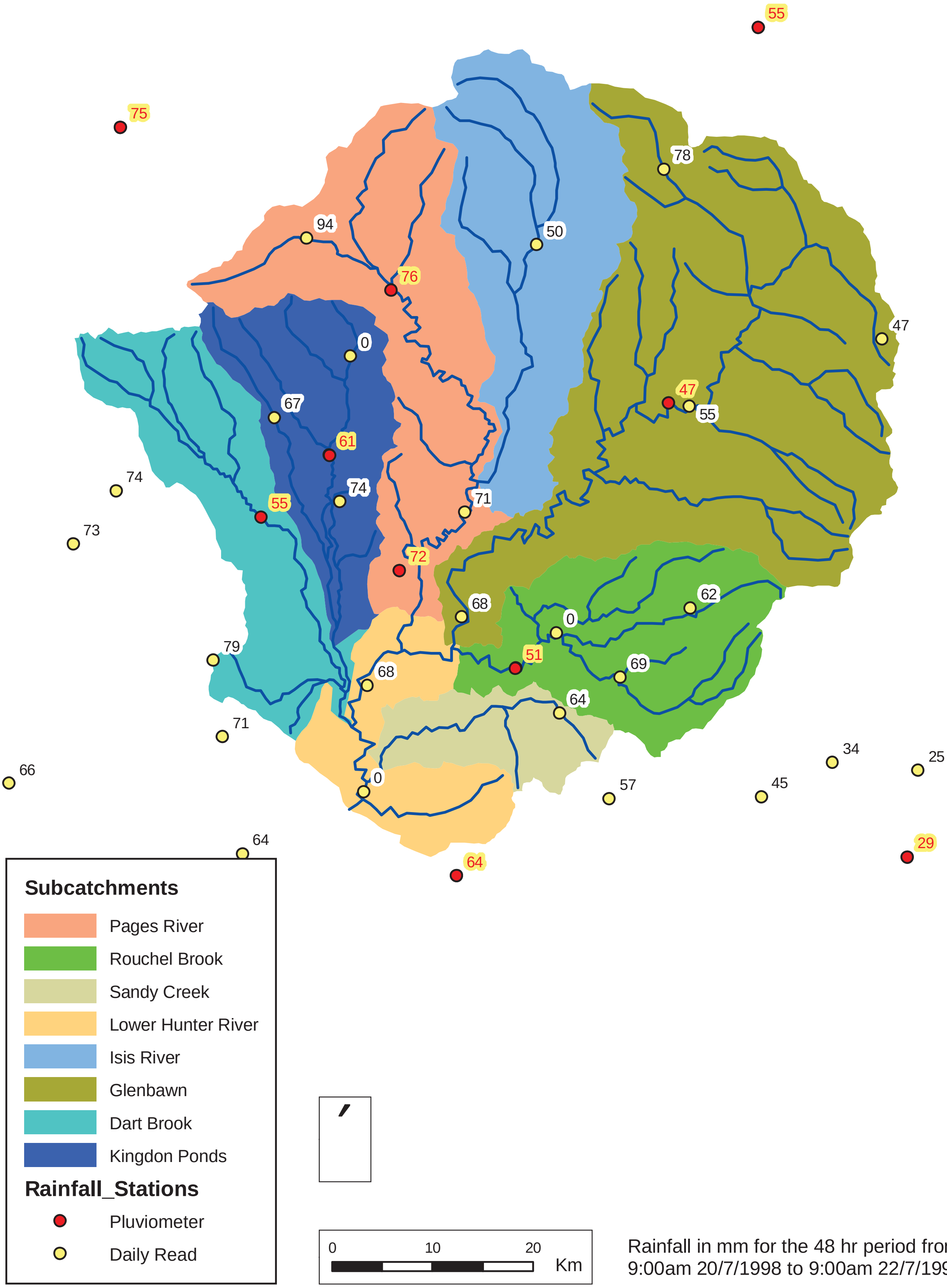
RAINFALL JANUARY 1996



Rainfall in mm for the 48 hr period from 9:00am 24/1/1996 to 9:00am 26/1/1996

FIGURE C18

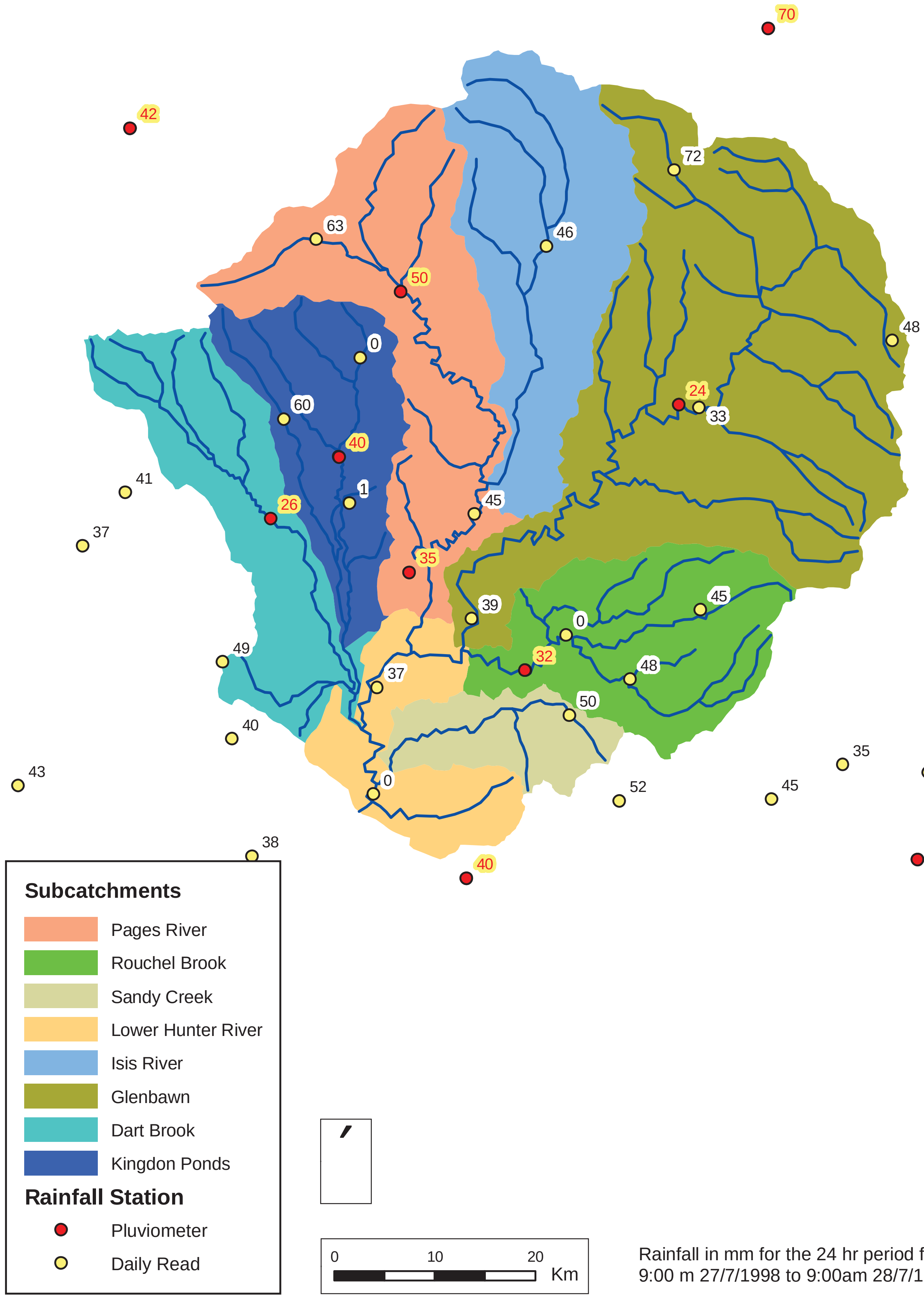
RAINFALL JULY 21st 1998



Rainfall in mm for the 48 hr period from 9:00am 20/7/1998 to 9:00am 22/7/1998

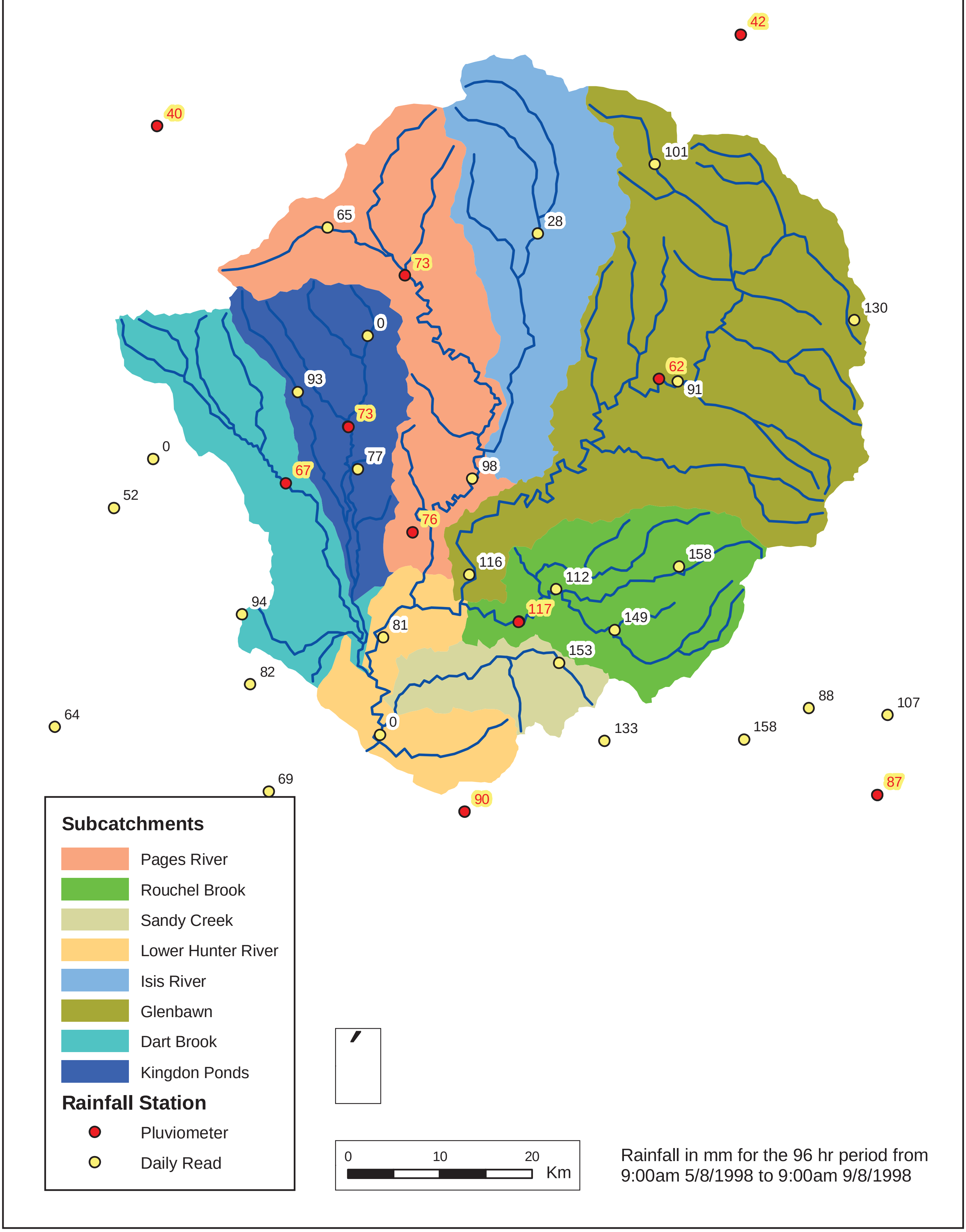
FIGURE C19

RAINFALL JULY 28th 1998



Rainfall in mm for the 24 hr period from 9:00 m 27/7/1998 to 9:00am 28/7/1998

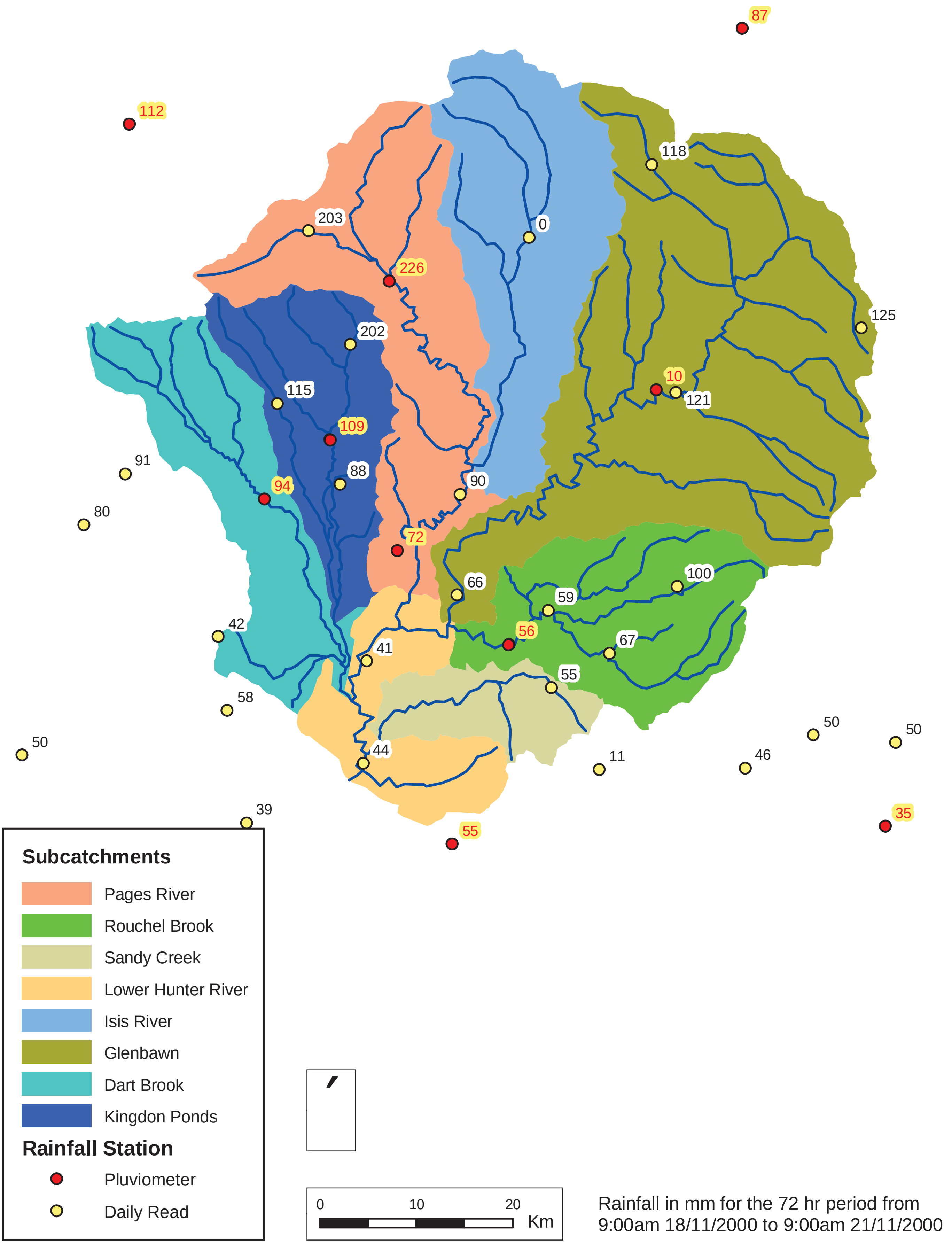
RAINFALL AUGUST 1998



Rainfall in mm for the 96 hr period from 9:00am 5/8/1998 to 9:00am 9/8/1998

FIGURE C21

RAINFALL NOVEMBER 2000



Rainfall in mm for the 72 hr period from 9:00am 18/11/2000 to 9:00am 21/11/2000

Appendix D

BOARDMAN PEASLEY PTY LIMITED

ACN 003 388 083 ABN 22 003 388 083

Consulting Surveyors & Civil Engineers

Director
R.A. Peasley Registered Land Surveyor MIS (NSW) CSA JP

Associates
G.V. Boardman Registered Land Surveyor MIS (NSW) CSA
N.J. Petersen Civil Engineer & Registered Land Surveyor MIS (NSW) CSA MIE (Aust)
P.E. Stone Registered Land Surveyor MIS (NSW) CSA

Muswellbrook Office (Head Office)
44 Market Street
PO Box 404, Muswellbrook N.S.W 2333

Phone: (02) 6543 3600
Fax: (02) 6542 5957
Website: www.boardmanpeasley.net.au
Email: ross@boardmanpeasley.net.au

Merriwa Office
100 Bettington Street
Merriwa N.S.W 2329
Phone/Fax: (02) 6548 2569
Mobile: 0429 433 600

8th August 2008

Our Reference: s4753-1

Your Reference:

Mr Richard Dewar - Director
WMA Water Pty Limited
Level 2, 160 Clarence Street
SYDNEY NSW 2000

Dear Sir,

RE: ABERDEEN FLOOD STUDY – GROUND TRUTHING SURVEY OF AERIAL PHOTOGRAPHY SITES 1 TO 9

As instructed by you we have carried out our survey of the nine (9) sites for the Aberdeen Flood Study requiring ground-truthing to verify their reduced levels.

We have used RTK GPS to carry out the surveys with the adoption of Permanent Mark numbered 120094 used as the base in most instances unless noted otherwise on the site plans. This mark has been verified by at least three (3) other established survey marks to establish survey integrity prior to commencement of each site/area. We note that the vertical precision of the survey is +/- 20 mm.

Please find herewith: -

(ii) Site plan of each of the nine (9) sites,
(iii) Our Memorandum of fees pursuant to your e-mail approval dated 23rd July 2008.

As requested we provide the following comments in relation to the levels and the accuracy of the related aerial photogrammetry:-

Site/Area #1
The levels shown in your photograph are generally lower than our levels surveyed in the order of around 0.25 to 0.35 metre.

Site/Area #2
The levels shown here are consistent with the aerial photograph provided within 50 mm which in our opinion is reasonably good given the lack of 'hard" surface to take levels upon.

Site/Area #3
Similar to site/area #1, the levels in the photograph are generally lower than our levels surveyed in the order of around 0.28 to 0.35 metre.

Site/Area #4
The levels shown here are consistent with the aerial photograph provided within 50 mm. There was a large amount of "hard' surface to take levels upon.

Site/Area #5
The levels shown here are consistent with the aerial photograph provided within 50 to 100 mm. Not sure if the levels shown had been rounded by the aerial photography, but generally match our survey.

Site/Area #6
The levels shown here are consistent with the aerial photograph provided within 50 to 100mm. Some of the levels shown on the aerial photograph at the edges of the road may have extended past the 'hard' surface as some appear to be different in the order of about 100 – 300 mm.

Site/Area #7
The levels shown here are consistent with the aerial photograph provided within 50 to 100mm. Then best correlation here is the kerb returns heading into Hall Street. The levels here are within 50mm.

Site/Area #8
The levels shown here are consistent with the aerial photograph provided within 50 to 150mm. Looking at the state of the road (Refer photograph on Sheet #8) if the correlating level was taken in pot hole or slightly off the formation then 100 to 150 mm could easily become evident.

Site/Area #9
The levels taken from the Aerial Photograph are generally higher than our survey in the magnitude of around 300 to 600 mm. This is more than expected. Given the correlation of the other eight (8) sites/areas I would have expected around 100 to 150 mm difference.

We trust our survey and comments in relation to each site have assisted in the modeling and reporting for the Aberdeen Flood Study.

Should you require any further assistance in this matter please contact us.

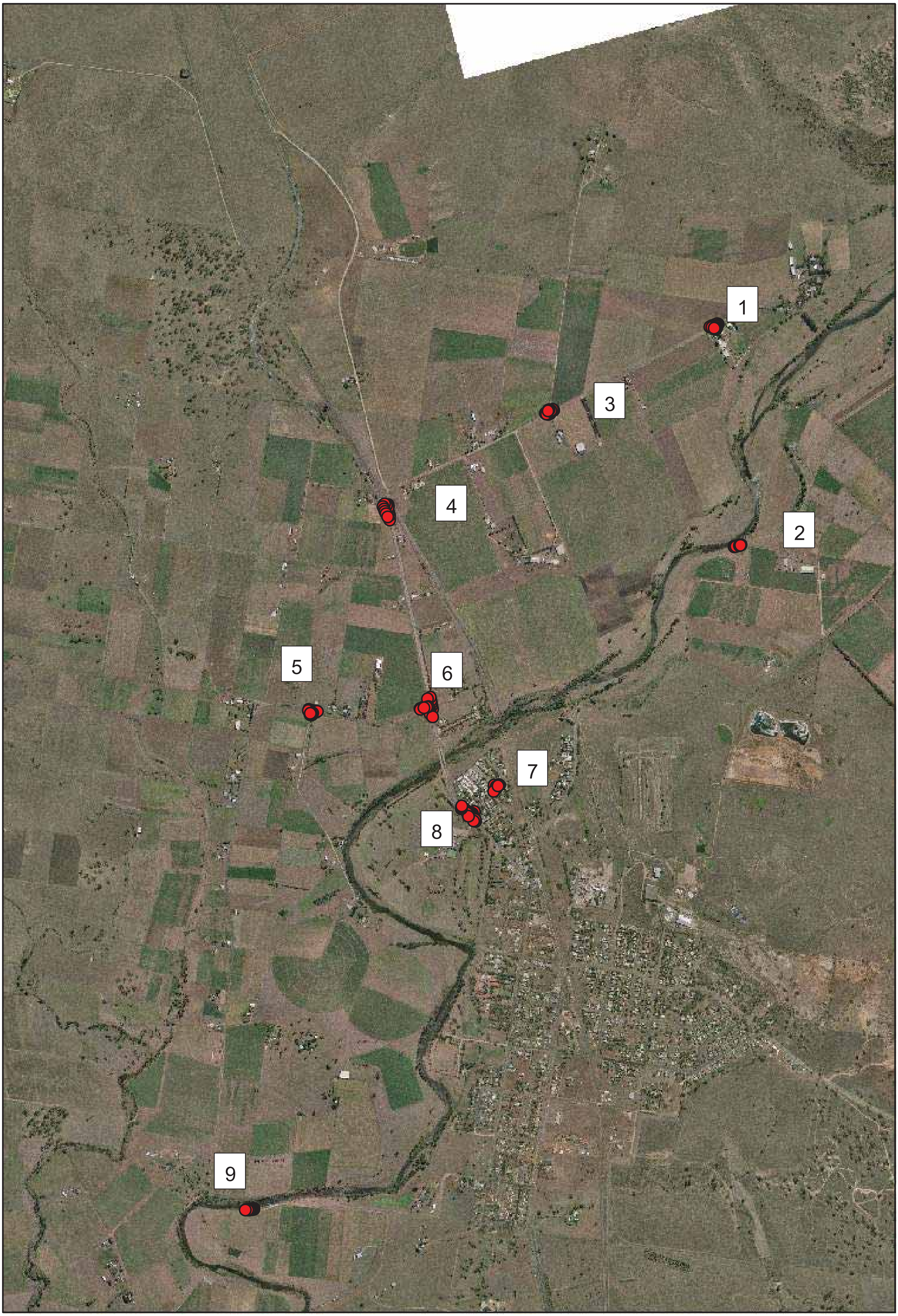
Yours faithfully
BOARDMAN PEASLEY PTY LIMITED

Ross Peasley

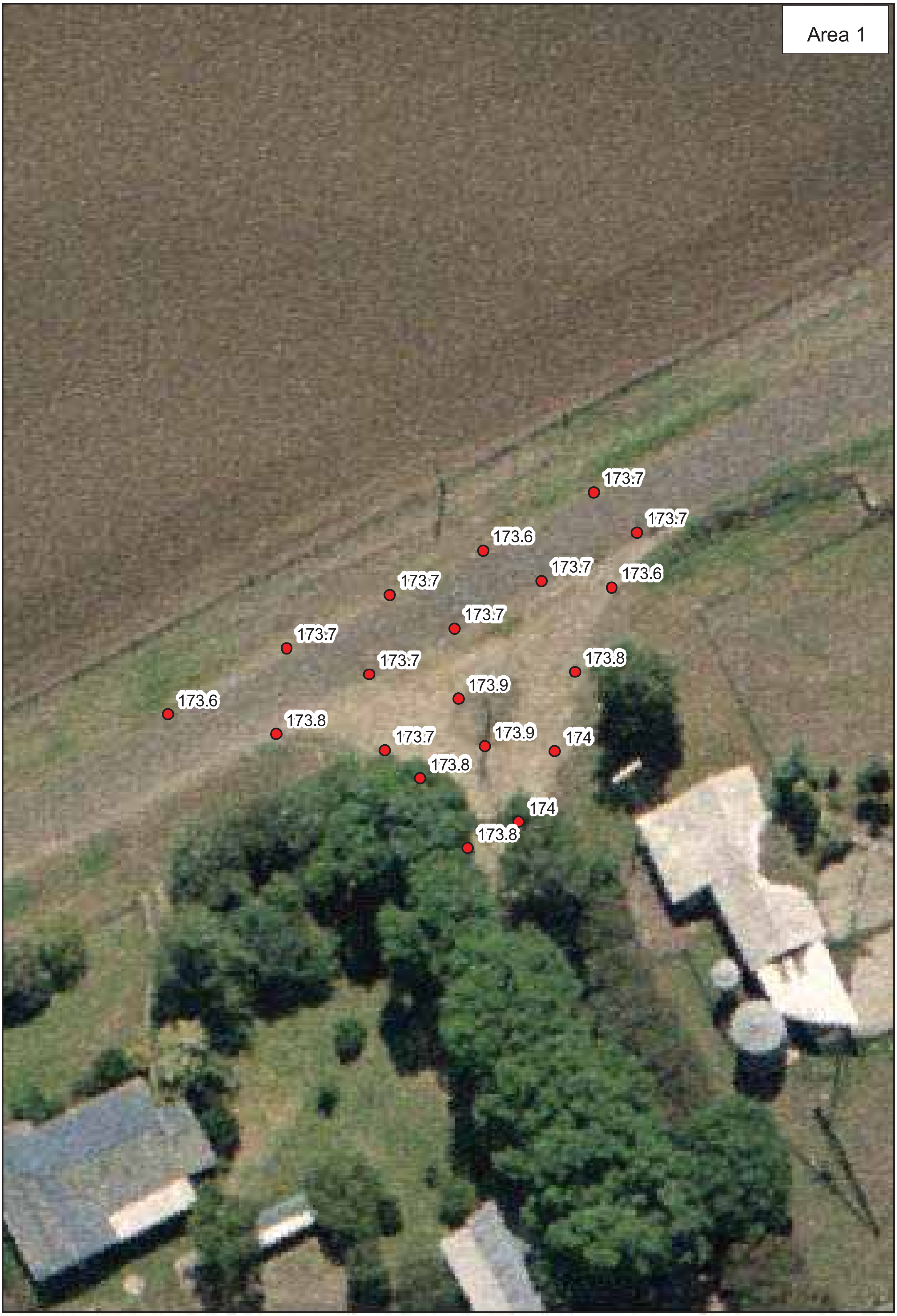
Surveyor Registered under the
Surveying Act, 2002, as amended.

COVER OF EXCELLENCE

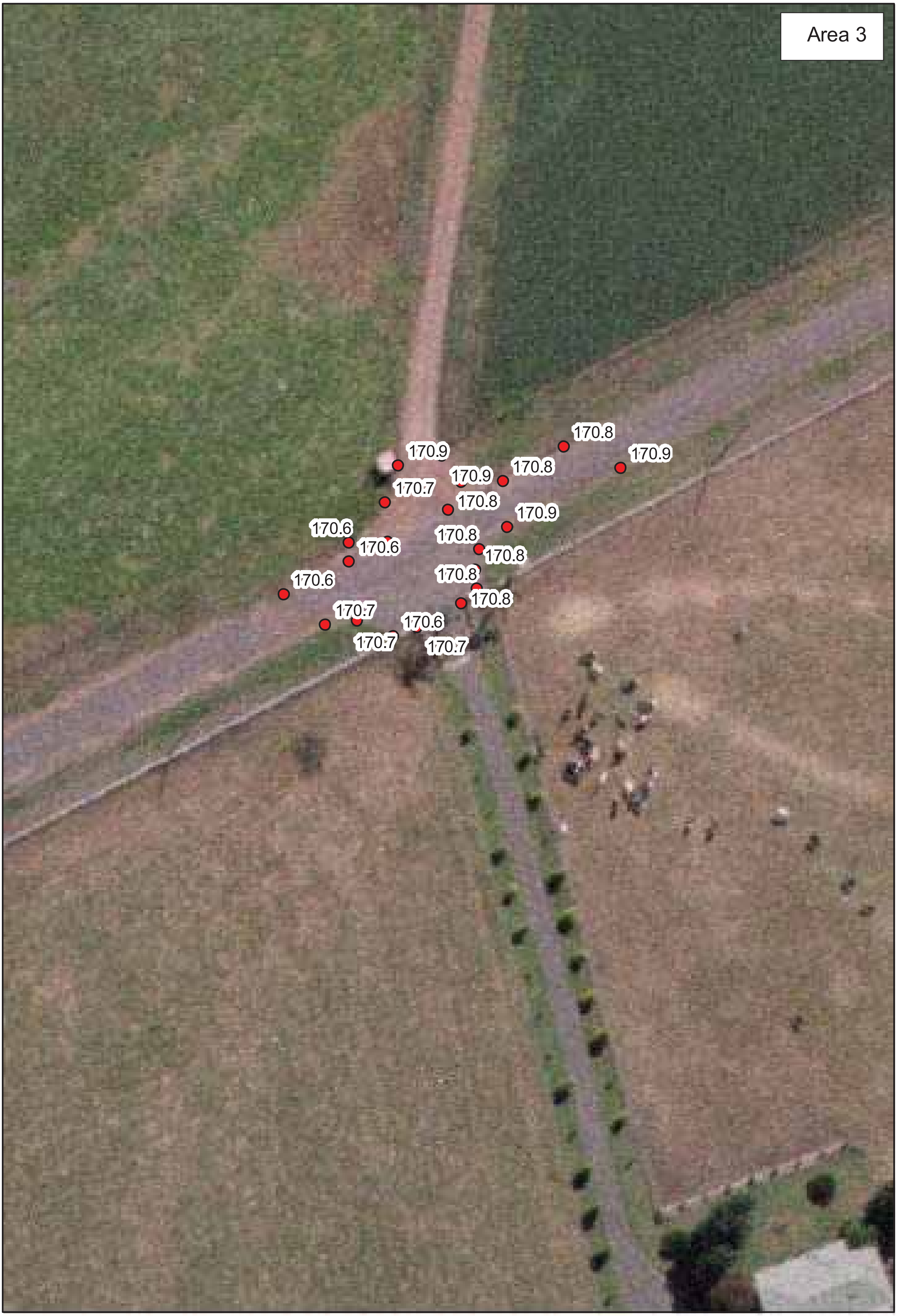
Liability limited by a Scheme approved under Professional Standards legislation



1
2
3
4
5
6
7
8
9


Area 1
173.7
173.7
173.6
173.7
173.6
173.7
173.7
173.7
173.7
173.8
173.9
173.6
173.8
173.7
173.9
174
173.8
174
173.8


Area 2
169.5
169.1
169
169.8
169
169.1
169.1



Area 3
170.8
170.9
170.9
170.8
170.9
170.7
170.8
170.9
170.6
170.8
170.6
170.8
170.6
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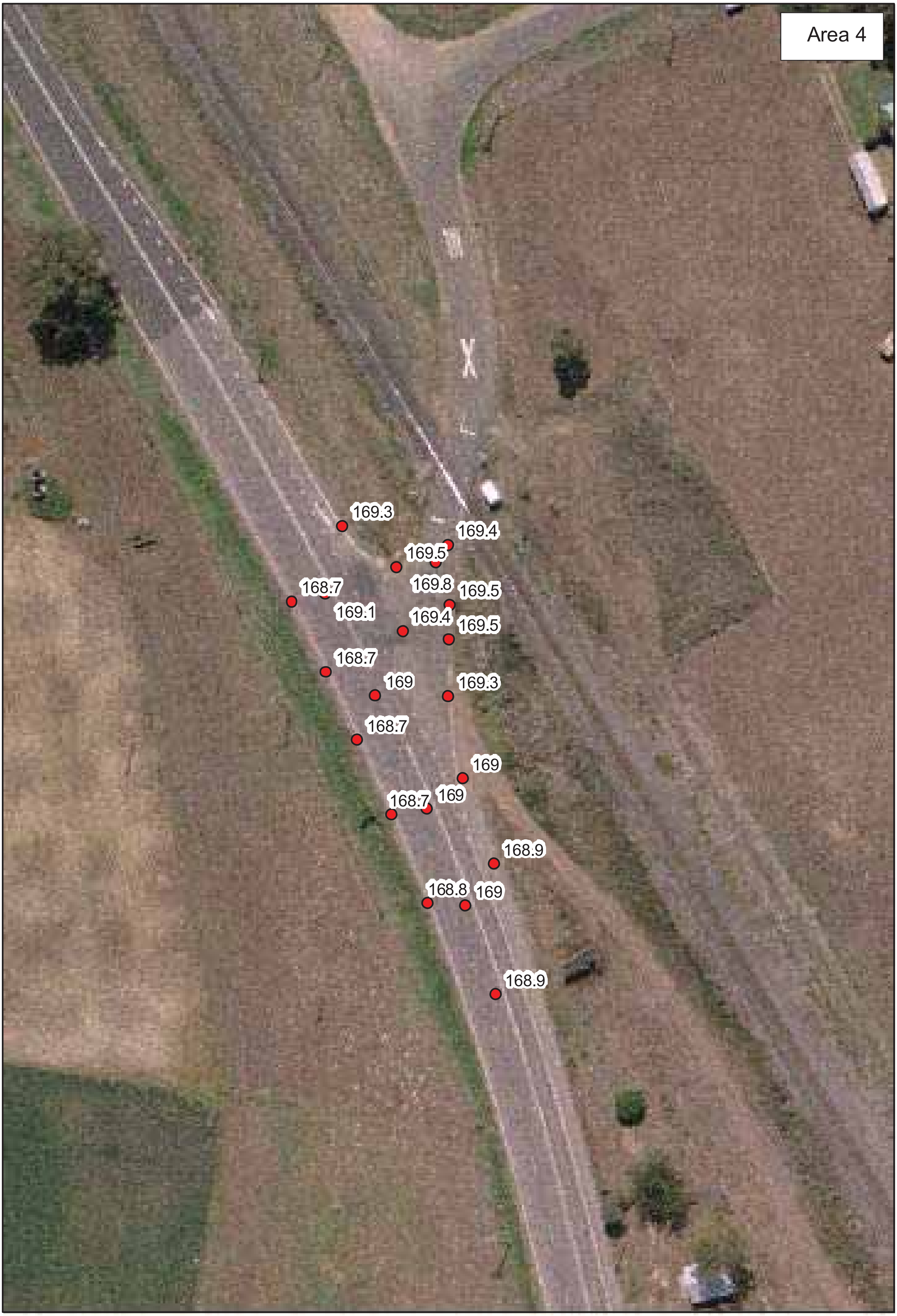
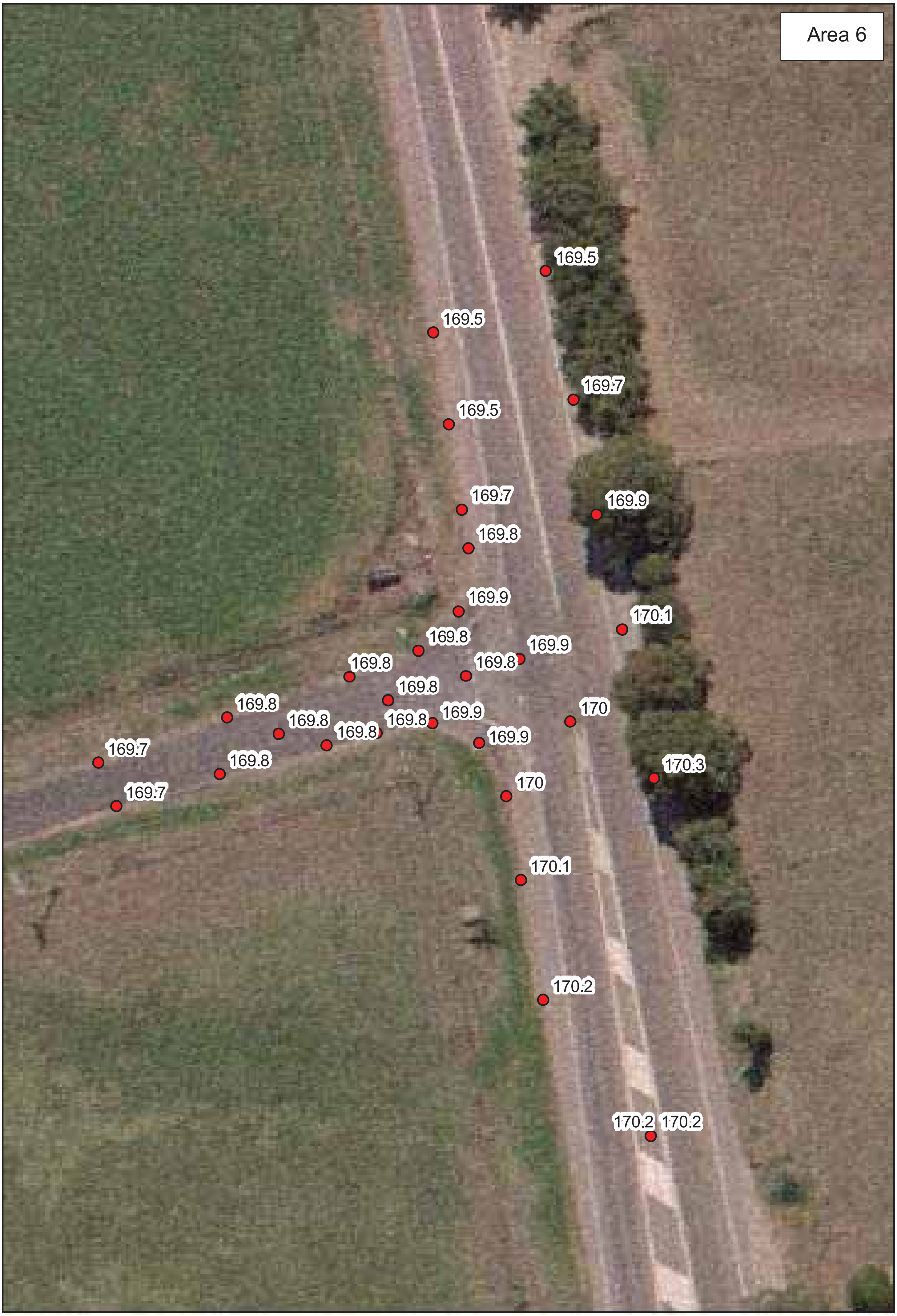
170.6
170.7
170.7

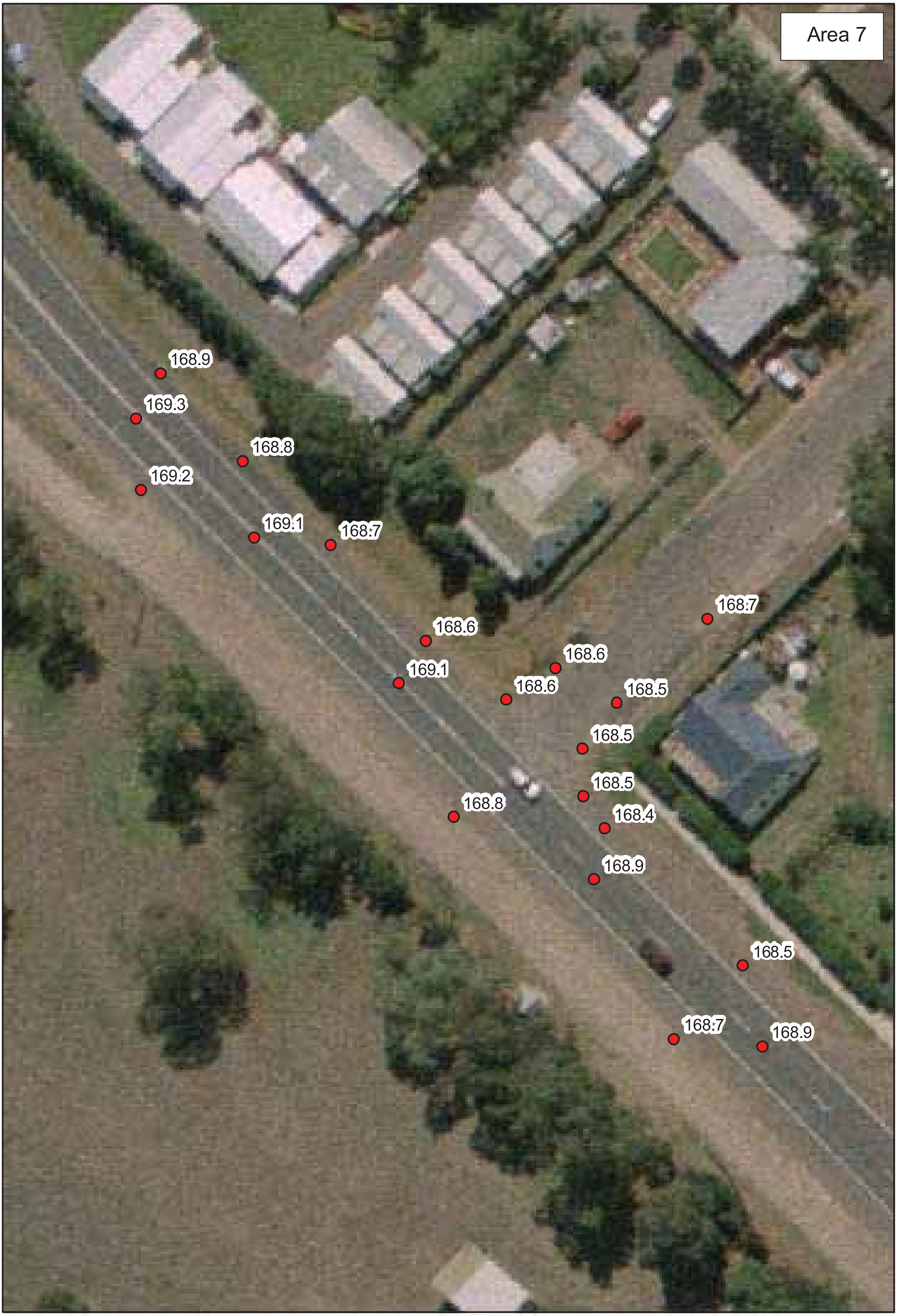


Area 4
169.3
169.4
169.5
168.7
169.8
169.5
169.1
169.4
169.5
168.7
169
169.3
168.7
169
168.7
169
168.9
168.8
169
168.9


Area 5
167
167
167.1
167.1
167
167
167.4
167.1
167.5
167.2
167.2
167.4
167
167.1
167.2
167
167.1
167.2
167
167.1
167.3



Area 6
169.5
169.5
169.7
169.5
169.7
169.9
169.8
169.9
170.1
169.8
169.9
169.8
169.8
169.8
169.8
170
169.8
169.8
169.9
169.8
169.8
169.9
169.7
169.8
170.3
169.7
170
170.1
170.2
170.2
170.2



Area 7
168.9
169.3
168.8
169.2
169.1
168.7
168.6
169.1
168.6
168.6
168.7
168.5
168.5
168.5
168.8
168.4
168.9
168.5
168.7
168.9


Area 8
168.4
168.5
168.3
168.4
168.4
168.4
168.4
168.6
168.6
168.5
168.3
168.3


Area 9
162.2
162.2
162.3
162.4
162.3
162.4
162.3
162.3
162.4
162.2
162.5
162.3
162.4
162.4
162.5