Proposed Expansion of Container Port Facilities in Botany Bay, NSW

Coastal Process and Water Resources Issues

Volume 1: Hydrologic and Hydraulic Studies

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Coastal Processes and Water Resources

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LIST OF ABBREVIATIONS

AEP	Annual Exceedance Probability
AHD	Australian Height Datum
AMG	Australian Mapping Grid
ARI	Average Recurrence Interval
AWRC	Australian Water Resources Council
BoM	Bureau of Meteorology
CBB	City of Botany Bay Council
DLWC	Department of Land and Water Conservation
DPWS	Department of Public Works and Services
EIS	Environmental Impact Statement
FPL	Flood Planning Level
GIS	Geographic Information System
GSDM	Generalised Short Duration Method
ha	hectare
IEAust	Institution of Engineers, Australia
IFD	Intensity Frequency Duration
km	kilometres
km^2	Square kilometres
LAT	Lowest Astronomical Tide
LEP	Local Environment Plan
LGA	Local Government Area
LIC	Land Information Centre
m	metre
m^2	Square metres
m^3	Cubic metres
m^3/s	Cubic meters per second
mAHD	Metres to Australian Height Datum
MHWL	Mean High Water Level

mm	millimetre
mm/hr	Millimeters per hour
m/s	metres per second
MSL	Mean Sea Level
NSW	New South Wales
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PWD	Public Works Department New South Wales
RAFTS	RAFTS proprietary hydrological software package
REP	Regional Environmental Plan
RTA	Roads and Traffic Authority
SES	State Emergency Service
SKM	Sinclair Knight Merz
SKP	Sinclair Knight Partners
SPC	Sydney Ports Corporation

PREFACE

This report is one of a series formed of three volumes. All reports have been prepared for Sydney Ports Corporation by Lawson and Treloar as part of water resources issues and coastal processes investigations undertaken to describe and quantify the potential impacts of proposed expansion of container port facilities in Port Botany, NSW.

Although each report is complete in itself, all reports draw upon the others for supporting information.

The reports prepared for this study are:-

1. Volume 1: Hydrologic and Hydraulic Studies (This Volume)
2. Volume 2: Water Quality Investigations
3. Volume 3: Waves, Currents and Coastal Process Investigations.

EXECUTIVE SUMMARY

ES1 Background

A flood impact assessment has been carried out for the proposed container port expansion at Port Botany. This report presents results of flood modelling studies undertaken for the catchments of Springvale Drain, Floodvale Drain and the Foreshore Beach Drains, as well as a discussion of the flood impacts of the proposed port development on the Mill Stream, based on existing modelling results for that catchment.

ES2 Catchment Overview

The Springvale Drain, Floodvale Drain and Foreshore Beach catchments are located within the City of Botany Bay Council Local Government Area at Banksmeadow. The principal land uses within the catchments include residential, industrial and paved roads/streets. Both Springvale and Floodvale Drains discharge stormwater to Botany Bay via Penrhyn Estuary whereas the Foreshore Beach Drains discharge directly to Botany Bay via pipe outlets along the Northern Foreshore Beach.

The Mill Pond catchment includes Centennial Park in the upper reaches to the north and the Botany Wetlands to the south. The Centennial Park ponds and the Botany Wetlands are connected by a stormwater drainage system and discharge to a channel referred to as the Mill Stream, which discharges to Botany Bay at a location parallel to the Parallel Runway of Kingsford Smith Airport.

ES3 Modelling Approach

Both hydrological and hydraulic modelling was undertaken. The hydrological model representing the existing catchment conditions covered the entire catchment for Springvale Drain, Floodvale Drain and the Foreshore Beach Drains. Design rainfall data was acquired from AR&R (1987) and design flood hydrographs for the 200 year, 100 year, 20 year and 5 year ARI events were generated using a rainfall-runoff routing model. A Probable Maximum Flood (PMF) hydrograph was also estimated using procedures recommended by the Bureau of Meteorology (BoM, 1994).

An hydraulic model was developed to determine flood levels before and after the proposed port expansion. The design flood hydrographs, determined using the hydrological model, were used as inputs to the hydraulic model along with assumed tailwater conditions in Botany Bay. Both low tailwater and high tailwater conditions were considered. The model results for the proposed port expansion were compared with the results for the existing conditions to identify any changes in flood levels.

ES4 Results

The results indicate that the proposed port expansion has no impact on flood levels in the Springvale Drain and Floodvale Drain catchments (i.e. the areas upstream of Foreshore Road) for events up to the 200 year ARI flood. There is a minimal increase in levels upstream of Foreshore Road in the Floodvale Drain catchment in the Probable Maximum Flood of the order of 0.02 m and is at the limit of the model's ability to predict change. The average recurrence interval of such an event is of the order of 1 in 10,000 years to 1 in 1,000,000 years. As a guide, the design flood adopted for planning purposes in New South Wales is usually the 1 in 100 year event. As such, the impact of the proposed port is zero at the flood planning level recurrence interval.

The results also indicate that the proposed port expansion has no impact on flood levels in the Foreshore Beach Drain catchments (i.e. the areas upstream of Foreshore Road) for events up to the 200 year ARI flood. There is a minimal increase in Bay water level near the outlets of Drains 1 and 2 in the Probable Maximum Flood of the order of 0.06 - 0.12 m. As outlined above, the average recurrence interval of such an event is of the order of 1 in 10,000 years to 1 in 1,000,000 years.

The proposed port expansion will not alter any part of the Mill Stream except for the construction of a small groyne at the western end of Foreshore Beach. Consequently the cross sections used in the hydraulic model remain the same for both the existing scenario and the proposed port development scenario. The groyne may improve the conveyance of the Mill Stream by preventing sand movement from the beach into the Stream. The modelling results show that there is no impact on water levels within the Mill Stream for the full range of flood events. The results show that no increases occur and therefore no impacts are observed upstream of Foreshore Road and the flood levels therefore do not increase in the Mill Pond catchment for the full range of design flood events.

1. INTRODUCTION

This report presents a flood impact assessment of the proposed container port expansion at Port Botany. Details of the proposed port expansion are provided in Appendix A.

The purpose of this study was to produce a range of surface runoff flood hydrographs at designated locations using design rainfall inputs, catchment topography and land use characteristics. A hydrologic model, RAFTS (WP Software, 1992), was used in this study for this purpose (Chapter 3). The surface runoff hydrographs were used as inputs to a hydraulic model, SOBEK (WL|Delft Hydraulics Laboratory, 2002), for the estimation of the levels and extents of various design floods (Chapter 4).

This report presents results of flood modelling studies undertaken for the catchments of the Springvale Drain, Floodvale Drain and the Foreshore Beach Drains. The flood impacts of the proposed port development on the Mill Stream were assessed based on existing modelling data for that catchment (Chapter 5) as well as hydraulic modelling of the lower reach of the channel. The locations of the catchments modelled for this study are shown in Figure 1.1.

The principal components of this study are: -

- collection, collation and review of documentation describing previous investigations related to the study area (Chapter 2),
- development of the hydrological model and estimation of design flood surface runoff hydrographs at designated locations to provide input to hydraulic flood modelling (Chapter 3),
- survey of relevant topographic and hydraulic structure features within the floodplain (Chapter 4)
- establishment of an hydraulic model and validation of this model to available data for the February 1990 flood event and estimation of design flood levels via hydraulic modelling for both the existing and proposed port scenarios (Chapter 4)
- an assessment of the potential impacts on flooding for the Mill Stream (Chapter 5), and
- discussion of the impacts of the proposed port development on flooding (Chapter 6).

A detailed glossary for the report can be found in Appendix B.

All levels are reduced to Australian Height Datum (AHD) unless otherwise stated. AHD is 0.925m above the Lowest Astronomical Tide Datum (LAT).

2. CATCHMENT AND FLOODPLAIN OVERVIEW

The Springvale Drain, Floodvale Drain and Foreshore Beach catchments are located at the heart of the City of Botany Bay Local Government Area (LGA) at Banksmeadow (Figure 1.1). The catchments have experienced significant urban and industrial development and modification since European settlement. The total area of the three catchments is 698 ha.

The Mill Pond Catchment (draining to the Mill Stream) has a much larger catchment of 1773 ha and covers a number of local government areas.

The catchment areas draining to the study area are shown in Table 2.1.

Table 2.1 Catchment Areas

Name	Springvale	Floodvale	Foreshore	Mill Pond
Area (ha)	241	118	339	1773

2.1 SPRINGVALE DRAIN AND FLOODVALE DRAIN

Springvale Drain and Floodvale Drain discharge stormwater to Botany Bay via Penrhyn Estuary (Figure 2.1). In large flood events flows from the two drains can exceed the capacity of the channels, spread overland and interact to form one large floodplain. In rare and extreme events significant channel and overland flows occur from the catchments and can result in the inundation of a large portion of Foreshore Road.

The principal land uses within the catchment include residential, industrial, recreational and paved roads/streets. The elevation varies from 0 mAHD in the south to 30 mAHD on top of the sand dunes of the Bonnie Doon Golf Course and Mutch Park. The overland slope ranges from 16% in the upper reaches to 0.01% near the foreshore area.

The northern part of the catchments contains mainly residential land use with some large open space areas such as Jellicoe Park, Mutch Park and part of the Bonnie Doon Golf Course.

The southern part of the catchments is mainly industrial and contains various large and small-scale developments including petroleum industries, food processing plants, chemical industries, shipping container areas and light industry. Botany Golf Course, located between Botany Road and Foreshore Road, is a significant open space. There are also substantial open areas in the vicinity of the channels between the Botany Freight Rail Line and McPherson Street. These open areas are largely low-lying wetlands that act as flood storage for overflows from the trunk drains. Except for the wetlands where the soils consist of peat, sandy peat and mud, the soils in both catchments are Botany Sands (NSW Department of Mineral Resources, 1983).

The total length of Floodvale Drain is 2.9 km, with approximately 2.1 km of closed conduit and 0.8 km of open channel. The total length of Springvale Drain is approximately 3.9 km, comprising of 2.5 km of closed conduit and 1.4 km of open channel (SKP, 1992 and SKM, 1996).

The channel is concrete lined in the lower parts of both Floodvale and Springvale Drains. In the upper parts, the channel varies from a grass-lined channel to areas that are rock-lined and sediment-lined. The floodplain consists of both urbanised areas (including industrial lands) with buildings and other obstructions as well as grassed overland areas. Flows to the channel are either via the stormwater drainage network connections or via overland flow paths.

Further details of the catchments can be found in Lawson and Treloar (2003).

2.2 FORESHORE ROAD DRAINAGE SYSTEM CATCHMENTS

The Foreshore Beach stormwater drainage system has five outlets along Foreshore Beach (Figure 2.1). Except for the open area between Botany Road and Foreshore Road, other parts of the catchment are residential and industrial. The total length of the five drains (closed conduits) is approximately 5.5 km (City of Botany Bay, undated).

Further details of the catchments for the drains can be found in Lawson and Treloar (2003).

2.3 MILL POND CATCHMENT

The Mill Pond catchment encompasses Centennial Park in the upper reaches to its north and the Botany Wetlands to its south. The Centennial Park ponds and the Botany Wetlands are connected by a stormwater drainage system and discharge to a channel referred to as the Mill Stream, which discharges to the Bay adjacent to the Parallel Runway of Kingsford Smith Airport.

Centennial Park consists of ten interconnecting ponds. These ponds receive stormwater runoff from a catchment of 590 hectares, which includes the former Showground site, Sydney Football Stadium, Sydney Cricket Ground (SCG) and parts of the suburbs of Randwick, Paddington, Bondi Junction and Queens Park. There are six direct stormwater inlets into the pond system from the surrounding suburbs. The ponds are connected by a series of internal stormwater drains, most of which run underground (Willing and Partners, 1999).

The Botany Wetlands are a cascade of eleven ponds and adjoining land forming a green corridor in the lower parts of the catchment of approximately 4 km in length and 56 ha in area. The wetlands play an important role in flood attenuation.

The natural landform of the Mill Pond catchment comprises rounded sand dunes and expanses of gentle slopes with local depressions and exposed water tables. Elevations vary from sea level to around 30 mAHD at the highest point on sandy dunes on the Eastlakes Golf Course and rise to about 70 mAHD near Centennial Park. The maximum level is approximately 100 mAHD at the north-eastern corner of the catchment along Botany Street, Bondi Junction.

Further details of the catchment can be found in Lawson and Treloar (2003).

Details of flood impacts on the Mill Stream are presented in Chapter 5.

3. HYDROLOGIC MODELLING

3.1 OVERVIEW

The Springvale Drain, Floodvale Drain and Foreshore Beach Drains catchments are typical urbanised catchments with residential, recreational, and industrial/commercial areas. A distinguishing feature of the catchments is the high infiltration capacity on pervious areas due to the sandy soils. According to previous studies (WRL, 1990, SKP, 1992 and SMEC, 1992), there are times when no surface runoff may occur from pervious areas if the duration of the storm event is short.

The following attributes were considered in the hydrological analysis of the catchments:

- Sub-catchment characteristics (such as area, slope, overland flow path length)
- Rainfall intensity-frequency-duration (IFD) relationships for the local area
- Land use (pervious and impervious areas), and
- Validation of the hydrologic model and critical flow duration estimation.

3.2 MODEL INPUT DATA AND PREVIOUS STUDIES REVIEW

Data was collected and collated from various sources. These included topographic data, meteorological data and other relevant data from previous studies.

3.2.1 Topographical Data and Catchment Delineation

Topographic data includes aerial photographs of eastern Sydney (Nos 179 and 181, Land and Property Information NSW, 2000) and Botany Bay (BPDP005A, Sydney Ports Corporation, 2000), the 1:2000 and 1:4000 orthophoto maps (LIC) and the Geological Series Sheet No 9130 (NSW Department of Mineral Resources, 1983).

Information on existing drains, (e.g. location, type, and size), for Springvale Drain, Floodvale Drain and the Foreshore Beach drains is available in Willing and Partners (1999), Sinclair Knight and Partners (1992), Sinclair Knight Merz (1996) and Water Research Laboratory (1990).

Based on the topographic features and land-use, these catchments were divided into 54 sub-catchments (24 for Springvale Drain, 11 for Floodvale Drain and 19 for the Foreshore Beach Drains). The sub-catchment layout is shown in Figure 3.1 and details of these sub-catchments are presented in Tables 3.1, 3.2 and 3.3.

Table 3.1 Springvale Drain Catchment Details

Subcatch ID	Area (ha)	Slope (%)	Impervious (ha)	Pervious (ha)	Impervious Fraction (%)
S1	20.29	0.30	12.17	8.12	60
S2	10.66	1.87	1.07	9.59	10
S3	27.57	1.68	2.76	24.81	10
S4	18.33	2.50	11.00	7.33	60
S5	9.26	3.34	4.63	4.63	50
S6	16.57	1.59	9.94	6.63	60
S7	13.54	1.23	6.77	6.77	50
S8	10.27	1.01	1.85	8.42	18
S9	7.36	0.69	3.68	3.68	50
S10	12.99	1.61	7.79	5.20	60
S11	13.67	1.16	8.89	4.78	65
S12	10.91	1.09	3.27	7.64	30
S13	5.36	0.14	1.61	3.75	30
S14	10.27	1.79	8.22	2.05	80
S15	14.17	0.62	2.83	11.34	20
S16	3.63	0.27	0.36	3.27	10
S17	3.42	2.37	2.74	0.68	80
S18	4.81	0.54	4.33	0.48	90
S19	2.98	0.43	2.68	0.30	90
S20	6.17	0.91	4.94	1.23	80
S21	3.46	0.04	2.77	0.69	80
S22	6.95	0.22	6.60	0.35	95
S23	7.09	0.04	0.71	6.38	10
S24	1.63	0.00	0.16	1.47	10
Sum	**241.36**	**-**	**111.77**	**129.59**	**-**

Table 3.2 Floodvale Drain Catchment Details

Subcatch ID	Area (ha)	Slope (%)	Impervious (ha)	Pervious (ha)	Impervious Fraction (%)
F1	18.99	2.59	13.29	5.7	70
F2	11.26	2.19	7.88	3.38	70
F3	15.86	0.65	7.93	7.93	50
F4	19.66	0.77	13.76	5.9	70
F5	10.14	1.04	8.11	2.03	80
F6	10.39	1.41	5.71	4.68	55
F7	10.7	0.51	7.49	3.21	70
F8	7.25	1.01	4.35	2.9	60
F9	5.14	0.16	3.6	1.54	70
F10	4.9	0.43	2.94	1.96	60
F11	3.78	0.09	0.38	3.4	10
Sum	**118.07**	**-**	**75.44**	**42.63**	**-**

Table 3.3 Foreshore Beach Drains Catchment Details

Subcatch ID	Area (ha)	Slope (%)	Impervious (ha)	Pervious (ha)	Impervious Fraction (%)
D1	10.53	4.07%	7.37	3.16	70
D2	23.93	0.57%	16.75	7.18	70
D3	13.68	1.02%	6.84	6.84	50
D4	15.86	0.43%	11.1	4.76	70
D4-1	14.91	0.66%	10.44	4.47	70
D5	20.97	2.77%	12.58	8.39	60
D6	14.76	1.05%	10.33	4.43	70
D7	21.01	1.13%	16.81	4.2	80
D8	15.48	0.37%	12.38	3.1	80
D9	10.18	0.26%	6.11	4.07	60
D10	28.85	0.29%	18.75	10.1	65
D11	23.64	0.37%	11.82	11.82	50
D12	14.12	1.51%	12.71	1.41	90
D13	9.06	1.25%	8.15	0.91	90
D14	16.26	2.42%	11.38	4.88	70
D15	19.16	1.39%	13.41	5.75	70
D16	22.22	0.67%	6.67	15.55	30
D17	27.88	0.38%	8.36	19.52	30
D18	16.8	0.40%	3.36	13.44	20
Total	**339.3**	**-**	**205.32**	**133.98**	**-**

The impervious portion of various subcatchments was adopted on the basis of recent aerial photography. Pervious areas include front and backyards on residential blocks and other open areas (golf courses or grassed open space). Impervious areas include streets, driveways, roofed areas and other paved surfaces. The total impervious area of these catchments was estimated as 56.9% of the total catchment area. This compares well with the average value of 61% for Springvale Drain and Floodvale Drain estimated by SKP (1992). The difference in the impervious fraction is likely to have arisen from the different methodologies used to calculate areas. This study utilised a GIS analysis of recent orthorectified aerial photography to estimate the impervious area for each subcatchment.

3.2.2 Meteorological Data

The average annual rainfall in this area is approximately 1100 mm and the highest rainfall usually occurs in the February – June period (Willing and Partners, 1999). Since the sizes of these catchments are reasonably small, the spatial rainfall distribution is likely to be relatively uniform. Therefore, for both historic and design flood events a uniform rainfall intensity was applied over the whole catchment. No rainfall area reduction factors were applied to the catchment.

Design Events

The Intensity Frequency Duration relationships (IFD) data for this area was derived from Australian Rainfall and Runoff (AR&R) Volume 2 (1987). The basic IFD parameters for the study area are listed in Table 3.4.

Table 3.4 IFD Parameters from AR&R (Volume 2, 1987)

Storm Duration	2 year ARI Intensity (mm/hr)	50 year ARI Intensity (mm/hr)
1 hour	41.9	87
12 hour	8.27	16.8
72 hour	2.55	5.19

For verification of IFD parameters, a comparison was undertaken for the design rainfall intensities derived for this study and those derived for other studies. Various design rainfall intensities used in previous studies are provided in Table 3.5.

Table 3.5 Comparison of IFD Parameters from Other Sources

	Botany Bay City Council (2000) Intensity (mm/h)		SMEC (1992) Intensity (mm/h)		SKP (1992) Intensity (mm/h)		WRL (1990) Intensity (mm/h)	
ARI	**2**	**50**	**2**	**50**	**2**	**50**	**2**	**50**
1 hour	41	84	NR	88	41.4	84.7	41.5	86
12 hour	8.1	16	NR	NR	8.1	16.3	8.0	16
72 hour	2.5	5.0	NR	NR	2.53	5.06	2.45	4.8

NR – Not reported

Table 3.5 shows there is a close similarity between the IFD parameters derived for this study (Table 3.4) and those reported in other sources.

Historic Event

The February 4 1990 flood event was used for the validation of the hydrological model (RAFTS) and the hydraulic model (SOBEK) (Section 3.4). This storm was chosen to validate the model as there was some peak flood water level data recorded during and after this storm. City of Botany Bay Council (D. Crompton *pers. comm*.) advise that no other significant storms have been recorded in the catchment for which there is suitable validation data.

Table 3.6 lists the rain gauges surrounding the catchment.

Table 3.6 Rainfall gauges in the Vicinity of the Study Area

Name	Code	Remark
Sydney Airport AMO	66037	6-min & 1-hr rainfall, complete record
Rockdale Bowling Club	66074	Daily rainfall, complete record
Maroubra RSL Bowling Club	66122	Daily rainfall, complete record
Marrickville	66036	Daily rainfall, complete record

Since the catchment response time is shorter than 24 hours, the daily rainfall gauges do not provide storm data in sufficient detail for catchment analysis. The 6-minute rainfall data from Sydney Airport AMO was the only useful data in terms of hydrological modelling of the historic storm. The data for the February 1990 rainfall event from the Sydney Airport Rainfall gauge was obtained from the Bureau of Meteorology's National Climate Centre and is shown in Figure 3.2.

3.2.3 Previous Studies

Sinclair Knight & Partners carried out a catchment management study for Floodvale Drain and Springvale Drains in 1992 (SKP, 1992). A MOUSE model (for hydrologic and hydraulic analysis) and a HEC-2 model (for hydraulic analysis) were developed and validated to the 4th February 1990 flood. The results (water levels and discharge values at various locations) from these studies were deemed useful for comparison purposes in the validation of the hydraulic model (SOBEK).

The Water Research Laboratory (WRL, 1990) carried out a flood study for the ORICA development site in the Springvale Drain catchment for which an ILSAX urban stormwater drainage model was developed. The IFD parameters (rainfall intensities, derived from AR&R) used in this study are provided in Table 3.5. The estimated peak discharges at selected locations for Springvale Drain obtained from ILSAX modelling are listed in Table 3.7.

Table 3.7 Springvale Drain Estimated Peak Discharges from ILSAX (after WRL, 1990)

Location	20 year ARI	50 year ARI	Time to Peak
Springvale Drain Culverts Under Sydenham to Botany Railway (ILSAX model: Pit A5)	17.4 m^3/s	19.3 m^3/s	40 min
Springvale Drain at ORICA Storage Tanks (ILSAX model: Pit A7)	22.4 m^3/s	26.4 m^3/s	45 min

Since ILSAX is a hydrological model, results for Springvale Drain listed in Table 3.7 are useful for comparison purposes in the validation of the hydrological model (RAFTS) in the Springvale catchment.

3.3 ESTABLISHMENT OF THE HYDROLOGICAL MODEL

The surface runoff hydrographs of the study area were estimated using the RAFTS (WP Software, 1992) rainfall-runoff modelling package. The sub-catchment layout as used in the RAFTS model is shown in Figure 3.1.

As outlined in Section 3.2.1, the RAFTS sub-catchments were established based on the 2m Land Information Centre (LIC) contour information and aerial photographs. Using the RAFTS utility, each sub-catchment was further divided to account for different initial/continuing rainfall loss rates for pervious/impervious areas (i.e. a split catchment modelling approach was adopted).

3.3.1 Model Validation

There are no flow gauges in the study area and hence the hydrological model could not be validated directly. A combined hydrologic/hydraulics approach was therefore adopted, where the hydraulic model was validated using input from the hydrologic model, thus indirectly validating the results of the hydrology model. The February 4 1990 storm event was used for this purpose. The 6-minute duration rainfall recorded at Sydney Airport was used for this purpose (Figure 3.2). As discussed in Section 3.2.2, a uniform areal distribution was assumed for the storm.

A selected list of RAFTS parameters are presented in Table 3.8.

Table 3.8 RAFTS Model Parameters

Ground Surface	Initial Loss (mm)	Continuing Loss (mm/hr)	Manning's n
Pervious	50	15	0.025
Impervious	1.0	1.0	0.010

The storage routing procedure in RAFTS was based upon topographic features of individual subcatchments, including slope, roughness (Manning's n), flow length and assumed velocity.

The initial and continuing loss for pervious areas reported in Table 3.8 are higher than the recommended values of AR&R (1998). As discussed in Chapter 2, the infiltration capacity of the Botany Sands is extremely high resulting in high initial and continuing losses to reflect the nature of the catchment. Both the Water Research Laboratory (1990) and Willing & Partners (1999) reported that for short duration storms, little runoff is generated from pervious areas.

In comparison, the initial and continuing losses were reported as 100 mm and 100 mm/hr respectively for open sandy areas in the RAFTS model developed by SMEC (1992) for the adjacent Mill Pond Catchment. Thus the values adopted for this study are conservative in comparison.

3.3.3 Comparison with Previous Studies

Table 3.9 presents a comparison of RAFTS peak discharges determined for this study with ILSAX peak discharges from WRL (1990) at the outlets of subcatchments S12 and S13 and the outlet of S9 (the subcatchment locations are shown in Figure 3.1).

Table 3.9 Comparison of RAFTS and ILSAX Peak Discharges

Location	20 year ARI (m^3/s)		50 year ARI (m^3/s)		Storm Duration (min)	
	RAFTS	ILSAX	RAFTS	ILSAX	RAFTS	ILSAX
RAFTS: Outlet of S9 ILSAX :Railway Pit A5	16.2	17.4	18.3	19.3	45	40
RAFTS: Outlets of S12 & S13 ILSAX: ORICA Storage Tanks, Pit A7	24.9	22.4	28.5	26.4	45	45

The outlet at subcatchment S9 coincides with the ILSAX model node at the Sydenham to Botany Railway (ILSAX Model Pit A5). At this location the RAFTS model reports slightly lower peak discharges than the ILSAX model (6-7% difference). This difference is within an acceptable limit of variation.

At the outlets of subcatchments S12 and S13, the peak discharges from RAFTS are slightly greater than the ILSAX model (7-10% difference). Given that the total catchment area draining to the outlets of the RAFTS subcatchments is larger than the catchment area draining to the ORICA storage tanks for the ILSAX model node A7, the slightly greater RAFTS model results are expected for this location.

It should be noted that the critical duration from the RAFTS model at these locations was found to be 2 hours. Storm results for the 45-min duration from the RAFTS model are listed in Table 3.9 because the '*time to peak*' of the ILSAX model is reported as being 40 and 45 minutes for the two nodes A5 and A7 respectively.

3.3.4 Comparison with the Rational Method

In addition to the above comparisons, the Urban Rational Method from AR&R Book VIII (1998) was also used for this purpose. The results of the Urban Rational Method and RAFTS are presented in Table 3.10. The '*time of concentration*' calculated for the Rational Method using the Kinematic Wave Equation for overland flow yielded a time of concentration of 45 minutes for catchment S9, 70 minutes for Springvale Drain and 50 minutes for Floodvale Drain.

Table 3.10 Comparison of RAFTS and Urban Rational Method Flood Peak (m^3/s)

Location	100 year ARI		50 year ARI	
	RAFTS	Rational	RAFTS	Rational
Outlet of Springvale	42.0	52.7	36.4	40.2
Outlet of Floodvale	27.4	32.1	23.7	26.7
Outlet of Catchment S9	23.7	31.0	20.4	26.9

Note: RAFTS results listed in this table correspond to 2-hr storms

Table 3.10 indicates that the Urban Rational Method tends to give higher estimates than the RAFTS model at all three locations. It is noted that the Urban Rational Method is strictly applicable to small catchments, usually a few hectares. In the present study, the sizes of the catchments are significantly larger than this (e.g. the total control area above subcatchment S9 is 1.34 km^2), and therefore the rational method is likely to overestimate peak flows. The results of the Urban Rational Method should therefore be regarded as indicative only.

3.4 HISTORICAL STORM EVENT MODELLING

As discussed in Section 3.1.4, the February 4 1990 storm event was adopted for validation purposes. The time series of 6 minute rainfall data for the February 4 1990 storm recorded at Sydney Airport was applied to all catchments in the RAFTS model to generate the validation hydrographs. An analysis of the rainfall data is provided in Table 3.11. As can be seen, the maximum 3 hour intensity of the February 1990 event is approximately a 2 year ARI event. The rarity of the February 4 1990 event increases for longer durations, up to six hours where the peak intensity was in the order of a 5 year ARI event.

Table 3.11 February 1990 Event - Rainfall Data (mm/hr)

Duration	Feb 90*	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI
6 min	72.0	124	156	175	199
30 min	35.8	61	79	90	104
1 hr	25.8	42	55	63	73
2 hr	22.8	27	35	40	46
3 hr	21.5	20	27	30	35
6 hr	16.8	13	17	19	22

*****At Sydney Airport AMO 66037

The RAFTS estimated peak discharges for the February 4, 1990 flood at various locations within the Springvale and Floodvale Drain catchments are presented in Table 3.12.

Table 3.12 February 1990 RAFTS Peak Discharges (m^3/s)

	Outlet	Botany Road	McPherson St	Railway
Springvale Drain	13.1	12.8 (S20/S21)	12.1 (S15/S16)	7.92 (S9)
Floodvale Drain	7.7	7.6 (F9/F10)	7.2 (F8)	4.7 (F3)

Note: The locations (catchment outlet) where the peaks are extracted are shown in brackets.

The estimated runoff hydrographs were utilised in the hydraulic model to match the flood levels and routed peaks at selected locations where the actual flood levels of the February 1990 event were reported. Further discussion of this can be found in Section 4.4.

3.5 DESIGN RAINFALLS

Design rainfall depths and temporal patterns for the 200 yr, 100 yr 20 yr and 5 yr ARI events were developed using standard techniques provided in AR&R Book II (1998). The basic parameters are listed in Table 3.4. Design storm rainfall intensities for the full range of storm frequencies and durations are presented in Table 3.13. As per the guidelines in AR&R, the Zone 1 temporal pattern was applied across the entire study area for the design storms. Due to the small area of the catchment, no spatial reduction factor was applied to the catchment.

Table 3.13 Design Rainfall Intensities (mm/h)

Frequency → Duration ↓	5 yr ARI	20 yr ARI	100 yr ARI	200 yr ARI	Probable Maximum Precipitation	
					A	B
15 min	109.45	141.48	183.33	201.59	620	590
30 min	79.23	103.74	135.89	150.03	450	430
45 min	64.42	85.05	112.20	124.19	380	370
1 hour	55.30	73.45	97.39	108.00	330	320
1.5 hour	42.38	56.17	74.35	82.39	250	240
2 hour	34.97	46.27	61.16	67.74	210	200
3 hour	26.58	35.09	46.29	51.24	160	150
6 hour	16.58	21.81	28.68	31.71	100	90
9 hour	12.61	16.54	21.71	23.98	-	-

The Probable Maximum Precipitation (PMP) was estimated using the Generalised Short Duration Method (BoM, 1994) recommended by the Bureau of Meteorology. Two rainfall intensities are presented in Table 3.13. The two values correspond to isohyets A and B defined for spatial distribution of PMP. The spatial distribution of isohyets A and B are shown in Figure 3.3.

3.6 DESIGN FLOWS

The design rainfall estimates were applied to the hydrologic model in order to predict design runoff hydrographs.

For the Probable Maximum Flood (PMF) estimates, a 0mm initial rainfall loss and a 1mm/hr continuing loss was assumed over the entire catchment as per the recommendation of AR&R (1998).

The critical duration was established from model runs for the 15min, 30min, 1hr, 2hr, 3hr, 6hr and 9hr storm events. The flows are reported in Table 3.14 at the catchment outlets of both Floodvale and Springvale Drains.

Table 3.14 Design Flows and Critical Durations

	Springvale Drain		Floodvale Drain		Drain 1		Drain 2		Drain 3 and 4		Drain 5	
Storm Event (ARI)	Critical Duration	Peak Flow (m^3/s)	Critical Duration	Peak Flow (m^3/s)	Critical Duration	Peak Flow (m^3/s)	Critical Duration	Peak Flow (m^3/s)	Critical Duration	Peak Flow (m^3/s)	Critical Duration	Peak Flow (m^3/s)
5 yr	2 hr	16.89	2 hr	14.61	1 hr	14.27	2 hr	17.71	1 hr	19.19	1 hr	20.68
20 yr	2 hr	23.59	2 hr	20.37	1 hr	18.98	2 hr	23.61	1 hr	25.75	1 hr	27.44
100 yr	2 hr	34.43	2 hr	27.67	1 hr	23.79	2 hr	29.96	1 hr	32.8	1 hr	34.82
200 yr	2 hr	40.16	2 hr	31.49	1 hr	26.57	2 hr	33.67	1 hr	36.75	1 hr	38.82
PMF	1 hr	163.41	1 hr	111.65	45 min	78.09	45 min	111.54	45 min	107.53	45 min	111.73

4. HYDRAULIC MODELLING

The purpose of hydraulic modelling is to identify any adverse impact on the local flooding behaviour due to the proposed port development. Data required for hydraulic modelling includes hydrological inputs, surveyed cross sections, hydraulic structure details, downstream boundary conditions and the design layout of the proposed container terminal (Appendix A). The area of assessment covers the lower reaches of the Mill Stream, Springvale Drain, Floodvale Drain, and the proposed port development area within the Bay in a single hydraulic model. The Foreshore Beach Drains were considered as hydrological inputs to the hydraulic model.

The extent of the modelled area is shown in Figure 4.1. Springvale Drain, Floodvale Drain and the Foreshore Beach Drains are discussed in this Chapter. The Mill Stream is discussed separately in Chapter 5.

4.1 PREVIOUS INVESTIGATIONS

Sinclair Knight & Partners carried out a catchment management study for both Floodvale and Springvale Drains (SKP, 1992). Design water levels and discharges at various locations for the 100 year, 20 year and 5 year ARI storms were produced. The results were estimated using the MOUSE and HEC-2 hydraulic models.

Further to the above study, Sinclair Knight Merz investigated the flood behaviour for the proposed co-generation plant upstream of McPherson Street (SKM, 1996). The proposed site is an area bounded by McPherson Street, Springvale Drain and Floodvale Drain. The modelling package used was also MOUSE and HEC-2, adapted from the 1992 study.

In addition to the above studies, Lawson and Treloar carried out a drainage study for ORICA's Botany site, east of the Sydenham – Botany Railway (L&T, 1998). A MOUSE model was developed for simulation of flood behaviour for the pipe drainage in the area. The site is beyond the current study area.

4.2 TOPOGRAPHIC SURVEY

For hydraulic modelling purposes, recent and detailed topographic survey data was acquired. Aerial photography available from SPC was used for photogrammetric analysis for parts of Springvale Drain, Floodvale Drain and the Foreshore Beach drains catchments. This photogrammetric analysis was carried out by AAM Surveys. The Bay area enclosed by the Parallel Runway, Foreshore Beach, Botany Road and the P&O container terminal at Port Botany was surveyed using depth soundings undertaken by SPC.

In addition to the photogrammetric analysis and soundings, detailed information such as channel cross sections, culvert dimensions, drainage pipe sizes and inverts and details for other hydraulic structures was also obtained. A ground survey was carried out by surveyors from SPC for this purpose. The survey sites were chosen based on modelling requirements and information on the system contained within available maps, aerial photographs and drainage plans of the area made available for this study by The City of Botany Bay Council.

4.3 MODEL ESTABLISHMENT

The impacts of the proposed port development were assessed using a fully dynamic one-dimensional hydraulic model, SOBEK (WL|Delft Hydraulics Laboratory, 2002). SOBEK is a finite difference model developed by WL|Delft Hydraulics Laboratory in The Netherlands. The model has been used worldwide and has been shown to provide reliable and robust results for simulation of flood behaviour in urban and rural areas through a number of applications. The solution scheme is capable of handling steep fronts, wetting and drying processes and subcritical and supercritical flow. The wide variety of hydraulic structures that the model can incorporate (weirs, roads, levees, culverts, bridges, etc) makes it a flexible and adaptable hydraulic analysis tool.

The drains were described as typical one-dimensional branches with cross-sections defining the channel geometry.

4.3.1 Model Set-up

The one-dimensional (1D) model set-up was carried out utilising the SOBEK GIS interface by registering an aerial photograph (the base map) in SOBEK. The location of surveyed cross sections, culvert inlets and outlets and the RAFTS subcatchment layout were also imported and marked on the base map. This was to ensure the model elements were located at the correct locations in the model layout.

In addition to the above considerations, site visits and a thorough review of reports of historical floods and available data was also carried out to aid the development of the hydraulic model. The model branch layout for the existing scenario is presented in Figure 4.1.

For the existing scenario, the model branches include Springvale Drain, Floodvale Drain and the Mill Stream (the open channel adjacent to the Parallel Runway downstream of Foreshore Road, see Chapter 5). The drains entering the bay from Foreshore Beach were not included as hydraulic elements in the model as the flow conditions at the downstream boundary (i.e. the water level in the Bay) were not likely to change due to flooding from the catchment. The flows from these drains were included in the model as hydrological inputs.

Under the proposed port development scenario (developed scenario), the model was modified to incorporate features shown on plans and cross sections in Appendix A. These plans and cross sections show a channel exit for Penrhyn Estuary around the proposed port running parallel to Foreshore Road. The cross sections show areas for intertidal sand and mud flats, saltmarsh and seagrass beds as part of the proposed habitat enhancement works. Cross sections from these drawings were incorporated into the hydraulic model developed scenario. The Mill Stream will not be modified from its existing state and therefore was not modified in the model.

The proposed development also includes a rail network that will cross the outlet channels of both Springvale and Floodvale Drains and run parallel to Foreshore Beach near Foreshore Road. A third rail crossing is proposed downstream of

Penrhyn Estuary over the proposed 130m wide channel. The location of the proposed railway is provided in the drawing *in* Appendix A.

The proposed railway crosses the Springvale Drain channel approximately 100m downstream of Penrhyn Road as a single span bridge with a minimum obvert of 2.5 mAHD. The bridge has been assumed not to encroach upon the existing cross section, and hence cause no reduction in the available flow conveyance.

The proposed rail crossing of Floodvale Drain occurs just downstream of Foreshore Road. Presently flow is carried from the catchment through a series of culverts under the golf course and Foreshore Road before discharging into Penrhyn Estuary downstream of Foreshore Road. These culverts are to be extended approximately 30m to pass under the proposed railway. Culvert sizes and grades are assumed to be the same as the existing construction.

The third crossing of Penrhyn Estuary is downstream of the confluence of Floodvale and Springvale Drains over the 130m wide channel (for locations refer to drawings in Appendix A). The approximate dimension of the proposed bridge support piers is 0.6m diameter, these piers will be located at 25m spacings across the channel.

A separate bridge for vehicular traffic is also proposed and will cross Penrhyn Estuary just downstream of the rail bridge. This bridge will have approximately 0.6m diameter piers spaced approximately every 30m across the channel as detailed in drawings *in* Appendix A.

Figure 4.2 shows the model layout for the proposed port development scenario,

4.3.2 Hydraulic Roughness

Without sufficient information to accurately validate the model roughness to the combined channel and over-bank roughness, a Manning's n of 0.03 was considered suitable for the drains. As such this was applied to Springvale Drain, Floodvale Drain, the Mill Stream and Penrhyn Estuary. This same roughness was applied to the channels representing the Bay area. This conservative value was adopted across all channel types as the model was developed for the purposes of a comparative study (i.e. to study change in pre and post development scenarios).

4.3.3 Model Boundaries

The model boundaries were located at the model extremities. The upstream boundaries were defined as discharge boundaries (Chapter 3), which were applied to the 1D branches of the model. The downstream boundary was defined as an Bay level boundary of 1.5 mAHD being the 100 year ARI storm surge level in Botany Bay (Manly Hydraulics Laboratory, 1992).

The downstream boundary adopted assumes that flooding in the catchment will occur at the same time as a severe ocean storm. However, this is a conservative approach and may 'drown out' the lower foreshore areas and thus conceal the potential impact of the proposed port development. Therefore, further analysis was carried out with a downstream boundary of 0.0m AHD at Botany Bay representing the expected water level in the Bay at the time of a flood. The full range of design storms was analysed using these two downstream tailwater conditions.

The upstream boundary discharge hydrographs for Floodvale Drain, Springvale Drain and the Foreshore Beach Drains were obtained from the RAFTS model (Chapter 3). The output hydrographs from the RAFTS catchments contained within the area of the hydraulic model were applied to the hydraulic model as inflow boundaries.

Since the change in the levels in Botany Bay in the pre and post development scenarios due to flooding from the local catchments was likely to be minimal, the downstream boundary conditions (Bay water level) for the Foreshore Beach pipes catchments will remain unchanged. This implies that the discharge capacity of the pipes would remain the same for the pre and post development scenarios. Consequently the flood behaviour for these pipe catchments would be unaltered.

The above approach was adopted in the expectation that the downstream tailwater levels would not change. If, in the unlikely event this hypothesis was not true, then full hydraulic modelling of the Foreshore Beach catchment would be undertaken. Chapter 6 outlines that the modelling indicates that this was not the case.

The discharge from the pipes that drain the catchments of Foreshore Beach (Figure 1.1) are included in the modelling. The total catchment flow from the Foreshore Beach Catchments was applied to the hydraulic model at the location of the outlet of the pipe. This approach is conservative, as the pipe capacity for each outlet is less than the peak catchment flow derived in the RAFTS model. However, the approach is justified, since the objective of the study is to determine the impact of the proposed development. Therefore the discharge input from the pipes remained the same for both existing and developed scenario modelling.

4.4 MODEL VALIDATION

The storm event of February 1990 was selected for validation purposes of the Floodvale Drain and Springvale Drain models. The inflow hydrographs to the SOBEK model were obtained from RAFTS using historic pluviograph data from Sydney Airport for the event (Section 3.2.2). The downstream boundary of the model was defined by an ocean water level time series (for the period 1^{st} - 5^{th} February 1990). The ocean water level for this period varied between -0.61 mAHD and 0.66 mAHD.

Reported flood levels and discharges at various locations were obtained from a previous flood study (SKP, 1992).

In the validation process, the hydraulic model parameters, such as channel roughness, were checked and a match was obtained between the recorded and modelled flood levels. The results of the validation are presented in Table 4.1.

Table 4.1 Model Validation Details – February 1990 Event

Location	Branch	Model WL (mAHD)	Road/Ground Level (mAHD)	Reported Depth, SKP (1992) (m)	Model Depth (col 3 – col 4) (m)	Difference (m)
McPherson St	Floodvale	3.24	2.71 - 3.01	0.30 - 0.40	0.23 - 0.53	0.07 - 0.13
Laport Chemical u/s of McPherson	Floodvale	3.28	3.22	Overtop right bank	Overtop right bank	N/A
Mobil Terminal	Floodvale	3.76	3.26	0.20	0.5	0.3
McPherson St	Springvale	3.30	3.39 – 3.70	No flow over street	No flow over street	N/A
Mobil Terminal	Springvale	5.00	4.70	Overtop	Overtop	N/A

Based on the available data and the modelling systems used for this study, the combined hydrologic/hydraulic validation produced reasonable results, thus giving confidence in the design modelling results.

4.5 DESIGN FLOOD BEHAVIOUR UNDER EXISTING SCENARIO

4.5.1 General

Design flood behaviour under the existing scenario was evaluated using the validated SOBEK model. Under the existing scenario, design flood inflow hydrographs, obtained from the RAFTS model, were applied to the hydraulic model representing the current catchment and downstream boundary conditions.

A full range of design events were considered including:

- PMF
- 200 year ARI,
- 100 year ARI,
- 20 year ARI and
- 5 year ARI.

Hydrographs of three storm durations for each ARI were used, those being the critical duration from RAFTS (Chapter 3) and a standard AR&R duration greater than and less than the critical duration. As discussed in Section 3.6, the critical duration estimated from RAFTS is 2 hours for all design events and 1 hour for the PMF except for some of the Foreshore Beach Drains, where the critical duration is 1 hour (45 minutes for the PMF). For the PMF, the durations considered included the 45min, 1hr and 2hr. For other design events, the durations considered included the 1hr, 2hr and 3hr.

The hydrographs from the storm durations spanning the critical duration were used in hydraulic modelling to capture the critical duration of the hydraulic model peak water levels, which is not necessarily the same as the critical duration estimated from RAFTS.

The design runs were carried out for both the elevated (1.5m AHD) and expected (0.0m AHD) downstream model boundaries (Section 4.3.3).

4.5.2 Results

Model results for the predicted flood behaviour under the existing scenario at significant locations are summarised in Table 4.2 (peak flow rates) and Table 4.3 (peak water levels). The reporting locations are shown in Figure 4.3 and were chosen to represent locations on each drain where a change in flood level may have a potential impact on urban areas.

Table 4.2: Summary of Peak Design Flow Rates – Existing Scenario

Location	Existing Scenario – Peak Flow Rates (m^3/s)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Inner Penrhyn Estuary **(A)**	146	149	28	27	24	24	18	18	14	14
Outer Penrhyn Estuary **(B)**	224	257	33	32	29	28	22	23	17	18
Floodvale Drain Upstream of Golf Course **(F)**	98	98	20	20	15	16	9	9	8	8
Springvale Drain Upstream of Penrhyn Road **(G)**	144	148	28	27	24	24	18	18	14	14

I – low tailwater condition
II – high tailwater condition

Table 4.3: Summary of Peak Design Flood Levels – Existing Scenario

Location	Existing Scenario – Peak Water Level (mAHD)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Inner Penrhyn Estuary **(A)**	1.22	1.64	0.59	1.50	0.55	1.50	0.47	1.50	0.40	1.50
Outer Penrhyn Estuary **(B)**	0.01	1.51	0.00	1.50	0.00	1.50	0.00	1.50	0.00	1.50
Floodvale Drain Upstream of Golf Course **(F)**	4.76	4.76	3.87	3.87	3.73	3.73	3.33	3.33	3.10	3.10
Springvale Drain Upstream of Penrhyn Road **(G)**	4.36	4.36	2.09	2.15	1.94	2.03	1.67	1.86	1.47	1.73

I – low tail water condition
II – high tail water condition

4.6 DESIGN FLOOD BEHAVIOUR UNDER PROPOSED PORT EXPANSION SCENARIO

4.6.1 General

The design flood behaviour under the proposed port expansion scenario was evaluated by comparing the flood behaviour at the same locations that were considered in the existing scenario. The SOBEK model was modified to represent the developed conditions as discussed in Section 4.3.1. The boundary conditions and input hydrographs were the same as for the existing scenario.

4.6.2 Results

Model results for the predicted flood behaviour under the proposed port development scenario at the same locations as for the existing scenario summarised in Table 4.4 (peak flow rates) and Table 4.5 (peak water levels). These locations are the same as for the existing scenario and are shown in Figure 4.3.

Table 4.4: Summary of Peak Design Flow Rates
Proposed Port Development Scenario

Location	Developed Scenario – Peak Flow Rates (m^3/s)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Inner Penrhyn Estuary **(A)**	146	149	28	27	24	24	18	18	14	14
Outer Penrhyn Estuary **(B)**	232	285	33	34	29	30	23	25	17	20
Floodvale Drain Upstream of Golf Course **(F)**	98	98	20	20	15	16	9	9	8	8
Springvale Drain Upstream of Penrhyn Road **(G)**	144	148	28	27	24	24	18	18	14	14

I – low tailwater condition
II – high tailwater condition

Table 4.5: Summary of Peak Design Flood Levels
Proposed Port Development Scenario

Location	Proposed Port Development Scenario Peak Water Level (mAHD)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Inner Penrhyn Estuary **(A)**	1.18	1.64	0.50	1.50	0.46	1.50	0.38	1.50	0.31	1.50
Outer Penrhyn Estuary **(B)**	0.13	1.57	0.00	1.50	0.00	1.50	0.00	1.50	0.00	1.50
Floodvale Drain Upstream of Golf Course **(F)**	4.78	4.78	3.87	3.87	3.73	3.73	3.33	3.33	3.10	3.10
Springvale Drain Upstream of Penrhyn Road **(G)**	4.36	4.36	2.09	2.15	1.94	2.03	1.67	1.86	1.47	1.73

I – low tailwater condition
II – high tailwater condition

Discussion and comparison of results can be found in Chapter 6.

5. FLOOD IMPACTS ON THE MILL STREAM CHANNEL

The impact on the Mill Stream was investigated for the reach of the stream downstream of Foreshore Road. The Mill Stream is a lined channel in this reach and runs parallel to the Parallel Runway of Sydney Airport. The channel discharges into Botany Bay at Foreshore Beach. The channel can be seen in Figure 1.1.

5.1 MODEL ESTABLISHMENT

The hydraulic model utilised to determine the flooding impacts in Floodvale Drain and Springvale Drain due to the proposed port development includes the Mill Stream. The input hydrograph for the Mill Stream was applied downstream of the weir (which is just upstream of Foreshore Road) that controls water levels in Engine Pond (part of the Mill Pond system). The top of the weir is set at 1.48 mAHD (SMEC, 1992).

The model cross sections were developed from survey data taken from the SPC soundings of the channel. Roughness values for the channel were assumed to have a Manning's 'n' of 0.03.

The proposed port expansion will not alter any part of the Mill Stream except for the construction of a small groyne at the western end of Foreshore Beach. Consequently the cross sections used in the hydraulic model remain the same for both the existing scenario and the proposed port development scenario. The groyne may improve the conveyance of the Mill Stream by preventing sand movement from the beach into the Stream.

5.1.1 Model Boundaries

The upstream boundary discharge hydrographs for the Mill Stream were derived from a previous flood study (SMEC, 1992). The peak flow data were provided at the concrete weir, which is located upstream of the South and Western Suburbs Ocean Outfall System (SWSOOS). The key parameters of the Mill Stream inflow hydrographs are presented in Table 5.1.

Table 5.1 Key Parameters for the Mill Stream Inflow Hydrographs

Design ARI	PMF	200 yr ARI	100 yr ARI	20 yr ARI	5 yr ARI
Peak flow (m^3/s)	219	57.1	49.2	32.5	22.5
Time to Peak (hr)	3	3	3	3	3

The peak PMF discharge was not provided in the SMEC (1992) flood study and was estimated using the Springvale Drain discharge data. The ratio of the 100 year ARI flow in Mill Pond Creek (at the weir) to the flow in Springvale Drain (at the Bay outlet) was 1.38. This ratio was then applied to the peak PMF flow from Springvale Drain to derive the peak flow at the weir for Mill Pond Creek. The same approach was adopted to derive a peak flow for the 200 year ARI event. Additionally, SMEC (1992) did not model the 20 year ARI, nor the 5 year ARI. To derive these values, those peak flows reported in SMEC (1992) for the 100 year, 50 year and 10 year ARI events were used to develop a regression relationship of flows and recurrence intervals. An estimate of the 20 year and 5 year ARI flows was made by extrapolation of this relationship.

As only the peak flow was reported by SMEC (1992), the hydrograph (discharge time series) was assumed to be triangular in shape, with the total volume of the hydrograph being equivalent to the excess rainfall and the time to peak discharge being the critical duration as determined by SMEC (1992).

Discharges from local catchments that border the Mill Stream channel modelled in RAFTS (Chapter 3) were included as lateral inflow boundaries to the model.

5.2 RESULTS

Model results for both the existing and the proposed port development scenarios are summarised in Tables 5.2, 5.3, 5.4 and 5.5.

Table 5.2: Summary of Peak Design Flow Rates for the Mill Stream Existing Scenario

Location	Peak Flow Rates (m^3/s)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Mill Stream at Botany Bay **(C)**	236	243	60	62	51	53	34	35	24	24
Mid Mill Stream **(D)**	220	232	57	57	49	50	32	33	22	23
Mill Stream at Foreshore Road **(E)**	219	224	57	57	49	49	32	32	22	22

I – low tailwater condition
II – high tailwater condition

Table 5.3: Summary of Peak Design Flood Levels for the Mill Stream Existing Scenario

Location	Peak Water Level (mAHD)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Mill Stream at Botany Bay **(C)**	0.12	1.59	0.01	1.50	0.00	1.50	0.00	1.50	0.00	1.50
Mid Mill Stream **(D)**	0.82	1.76	0.10	1.51	0.07	1.51	0.03	1.50	0.02	1.50
Mill Stream at Foreshore Road **(E)**	1.13	1.84	0.19	1.52	0.15	1.52	0.07	1.51	0.04	1.50

I – low tailwater condition
II – high tailwater condition

Table 5.4: Summary of Peak Design Flow Rates for the Mill Stream Proposed Port Development Scenario

Location	Peak Flow Rates (m^3/s)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Mill Stream at Botany Bay **(C)**	236	243	60	62	51	53	34	35	24	24
Mid Mill Stream **(D)**	220	232	57	57	49	50	32	33	22	23
Mill Stream at Foreshore Road **(E)**	219	224	57	57	49	49	32	32	22	22

I – low tailwater condition
II – high tailwater condition

Table 5.5: Summary of Peak Design Flood Levels for the Mill Stream Proposed Port Development Scenario

Location	Peak Water Level (mAHD)									
	PMF		200 yr		100 yr		20 yr		5 yr	
	I	II	I	II	I	II	I	II	I	II
Mill Stream at Botany Bay **(C)**	0.12	1.59	0.01	1.50	0.00	1.50	0.00	1.50	0.00	1.50
Mid Mill Stream **(D)**	0.82	1.76	0.10	1.51	0.07	1.51	0.03	1.50	0.02	1.50
Mill Stream at Foreshore Road **(E)**	1.13	1.84	0.19	1.52	0.15	1.52	0.07	1.51	0.04	1.50

I – low tailwater condition
II – high tailwater condition

Discussion and comparison of results can be found in Chapter 6.

6. DISCUSSION

6.1 OVERVIEW OF IMPACT OF PROPOSED DEVELOPMENT

A comparison of peak flood levels for the existing and proposed port development scenarios is provided in Tables 6.1 (for a downstream boundary condition of 1.5m AHD) and Table 6.2 (for a downstream boundary condition of 0.0m AHD).

As outlined in Section 4.3.3, the impact analysis was carried out for two tailwater (downstream boundary) conditions. The high tailwater level of 1.5m AHD represented a severe ocean storm condition, which when combined with catchment flooding produced a certain flood behaviour. The low tailwater level of 0.0m AHD was assessed to determine if there was an impact on peak water levels, where the potential impact to Penrhyn Estuary or other parts of the proposed development will not be drowned out by the high water level in Botany Bay.

6.2 SPRINGVALE AND FLOODVALE DRAIN

The results show that the proposed port development will not have an impact on flood levels in Floodvale Drain, Springvale Drain catchments up to the 200 year ARI.

The proposed habitat enhancement works in Penrhyn Estuary do not reduce flow conveyance in this area. The channel at the outlet to the estuary, being 130 metres wide and dredged with a variable depth, does not constrict the flow such that no increase in water levels upstream is observed up to the 200 year ARI.

A minor increase of 0.02 m would be observed in the Floodvale Drain catchment (Table 6.1, Site F) for the Probable Maximum Flood in both the low and the high tailwater condition. This event is the most extreme event likely to occur with an estimated average recurrence interval of 1 in 10,000 to 1,000,000 or a 0.0001 - 0.000001% chance of occurring in any one year (AR&R, 1998). Thus, this very small impact has a very low chance of occurrence and is at the limit of the model's ability to predict change.

6.3 PENRHYN ESTUARY

Flood levels in Penrhyn Estuary (Table 6.2, Site A) would be reduced under the low tailwater condition by between 0.04 - 0.09 m under the proposed port expansion scenario due to the habitat enhancement works.

No changes in flood levels would be observed in the high tailwater condition due to the proposed port expansion.

6.4 FORESHORE BEACH DRAINS

The discharge capacity of the pipes draining the Foreshore Beach catchments is affected by water levels in the Bay. As the proposed development does not change water level (the peak water level and the water level time series remains the same) in the Bay up to the 200 year ARI event, there is no impact on the discharge capacity of these pipes. The flood levels within the Foreshore Beach catchments are therefore unaffected. This analysis is based on the assumption that the pipe lengths will remain unchanged for the proposed developed condition and hence pipe losses will remain the same, with the downstream water level being the only factor that could change under the proposed port expansion scenario.

In the Foreshore Beach drain modelling, the flow generated from the drain catchments was applied directly to the hydraulic model. This approach is conservative and does not take into account the significant hydraulic controls upstream of the drain outlets.

The approach is justified since this study is a comparative study where the difference in flood behaviour is considered for pre and post development scenarios. The flow from the Foreshore Beach drains remains the same for the two development scenarios.

The approach adopted also demonstrates that there will be no impact on water levels in the Bay up to the 200 year ARI even if hydraulic improvements are undertaken by increasing the flow capacity of the pipes draining the catchment. That is, even if the hydraulic improvements allow the full hydrograph to be translated through to the beach (i.e. no upstream controls attenuate flow), there will be no change in the Bay levels as represented in the present modelling.

The proposed railway bridges and the vehicle bridge do not impact the water levels in the Bay near the outlet of Drains 1 and 2 up to the 200 year ARI if the flow regime under these bridges is maintained as under existing conditions. The railway bridge adjoining the vehicle bridge will result in a very small increase in flood levels in this area (Site B in Tables 6.1 and 6.2) at the Probable Maximum Flood (0.06 m for the high tailwater condition and 0.12 m for the low tailwater condition). As outlined above, this event is the most extreme event likely to occur with an estimated average recurrence interval of 1 in 10,000 to 1,000,000 or a 0.0001 - 0.000001% chance of occurring in any one year (AR&R, 1998). Thus, this very small impact has a very low chance of occurrence and is at the limit of the model's ability to predict change.

6.5 MILL STREAM

Flooding upstream of the SWSOOS carrier is controlled by the presence of the weir (crest level at 1.48m AHD) upstream of Foreshore Road and is generally due to catchment flooding rather than coastal elevated water level flooding. However, under extreme ocean conditions, such as assumed for the 100 year ARI event, flooding from Botany Bay is possible. Since there is no increase in the Bay levels due to the proposed development, there would be no impact on flood levels upstream of the SWSOOS carrier due to the proposed port development.

For the low tailwater conditions the SWSOOS weir controls the flooding upstream of the weir, which would be catchment flooding. Therefore the proposed port expansion (downstream of the weir) would not have any impact on flooding in areas upstream of the weir.

Table 6.1: Comparison of Peak Flood Levels – Existing and Proposed Port Development Scenarios
High Tail Water Condition (all flood levels are in mAHD)

ARI	5 Year			20 year			100 year			200 year			PMF		
Location	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)
Inner Penrhyn Estuary (A)	1.50	1.50	0.00	1.50	1.50	0.00	1.50	1.50	0.00	1.50	1.50	0.00	1.64	1.64	0.00
Outer Penrhyn Estuary (B)	1.50	1.50	0.00	1.50	1.50	0.00	1.50	1.50	0.00	1.50	1.50	0.00	1.51	1.57	0.06
Mill Pond Creek at Bay (C)	1.50	1.50	0.00	1.50	1.50	0.00	1.50	1.50	0.00	1.50	1.50	0.00	1.59	1.59	0.00
Mid Mill Pond Creek (D)	1.50	1.50	0.00	1.50	1.50	0.00	1.51	1.51	0.00	1.51	1.51	0.00	1.76	1.76	0.00
Upper Mill Pond Creek (E)	1.50	1.50	0.00	1.51	1.51	0.00	1.52	1.52	0.00	1.52	1.52	0.00	1.84	1.84	0.00
Upstream of Golf Course – Floodvale Drain (F)	3.10	3.10	0.00	3.33	3.33	0.00	3.73	3.73	0.00	3.87	3.87	0.00	4.76	4.78	0.02
Upstream of Penrhyn Road – Springvale Drain(G)	1.73	1.73	0.00	1.86	1.86	0.00	2.03	2.03	0.00	2.15	2.15	0.00	4.36	4.36	0.00

Table 6.2: Comparison of Peak Flood Levels – Existing and Proposed Port Development Scenarios
Low Tail Water Condition (all flood levels are in mAHD)

ARI	5 Year			20 year			100 year			200 year			PMF		
Location	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)	Existing	Developed	Increase in Water Level (m)
Inner Penrhyn Estuary (A)	0.40	0.31	-0.09	0.47	0.38	-0.09	0.55	0.46	-0.09	0.59	0.50	-0.09	1.22	1.18	-0.04
Outer Penrhyn Estuary (B)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.13	0.12
Mill Pond Creek at Bay (C)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.00	0.12	0.12	0.00
Mid Mill Pond Creek (D)	0.02	0.02	0.00	0.03	0.03	0.00	0.07	0.07	0.00	0.10	0.10	0.00	0.82	0.82	0.00
Upper Mill Pond Creek (E)	0.04	0.04	0.00	0.07	0.07	0.00	0.15	0.15	0.00	0.19	0.19	0.00	1.13	1.13	0.00
Upstream of Golf course – Floodvale Drain (F)	3.10	3.10	0.00	3.33	3.33	0.00	3.73	3.73	0.00	3.87	3.87	0.00	4.76	4.78	0.02
Upstream of Penrhyn Road – Springvale Drain(G)	1.47	1.47	0.00	1.67	1.67	0.00	1.94	1.94	0.00	2.09	2.09	0.00	4.36	4.36	0.00

7. RECOMMENDATIONS AND CONCLUSIONS

The following conclusions can be drawn from the study results:

- For up to the 200 year ARI event, the proposed port development will not have any adverse impact on local flood behaviour upstream of Penrhyn Road, Botany Road or Foreshore Road. A minor impact (0.02 m) will be observed in the Floodvale Drain catchment during extremely rare events.
- For both the 100 year ARI and PMF floods, the proposed port development will not cause an increase in flood levels within Penrhyn Estuary.
- There will be no impact on the Foreshore Beach catchments drained by the pipe culverts under Foreshore Road up to the 200 year ARI event. A minor impact (0.12 m) will be observed at the location where these drains discharge to the Bay (i.e. in the 130 m wide channel) during extremely rare events. During the detailed design phase for the proposed port expansion any drains that discharge to Foreshore Beach that need to be moved or extended will need to undergo a drainage analysis to ensure that additional losses caused by the lengthening or redirection of the pipes does not adversely affect pipe conveyance. All Foreshore Beach stormwater drainage pipes that remain unchanged will not be affected by the proposed port development up to the 200 year ARI, leading to no change in the water levels at the inlet of the pipe culverts for the vast majority of events.
- The concrete weir at the SWSOOS has a level of 1.48 mAHD and will be overtopped under both existing and developed scenarios in the event of a 100 year ARI flood. The proposed port development will not increase flood levels in this region.

8. REFERENCES

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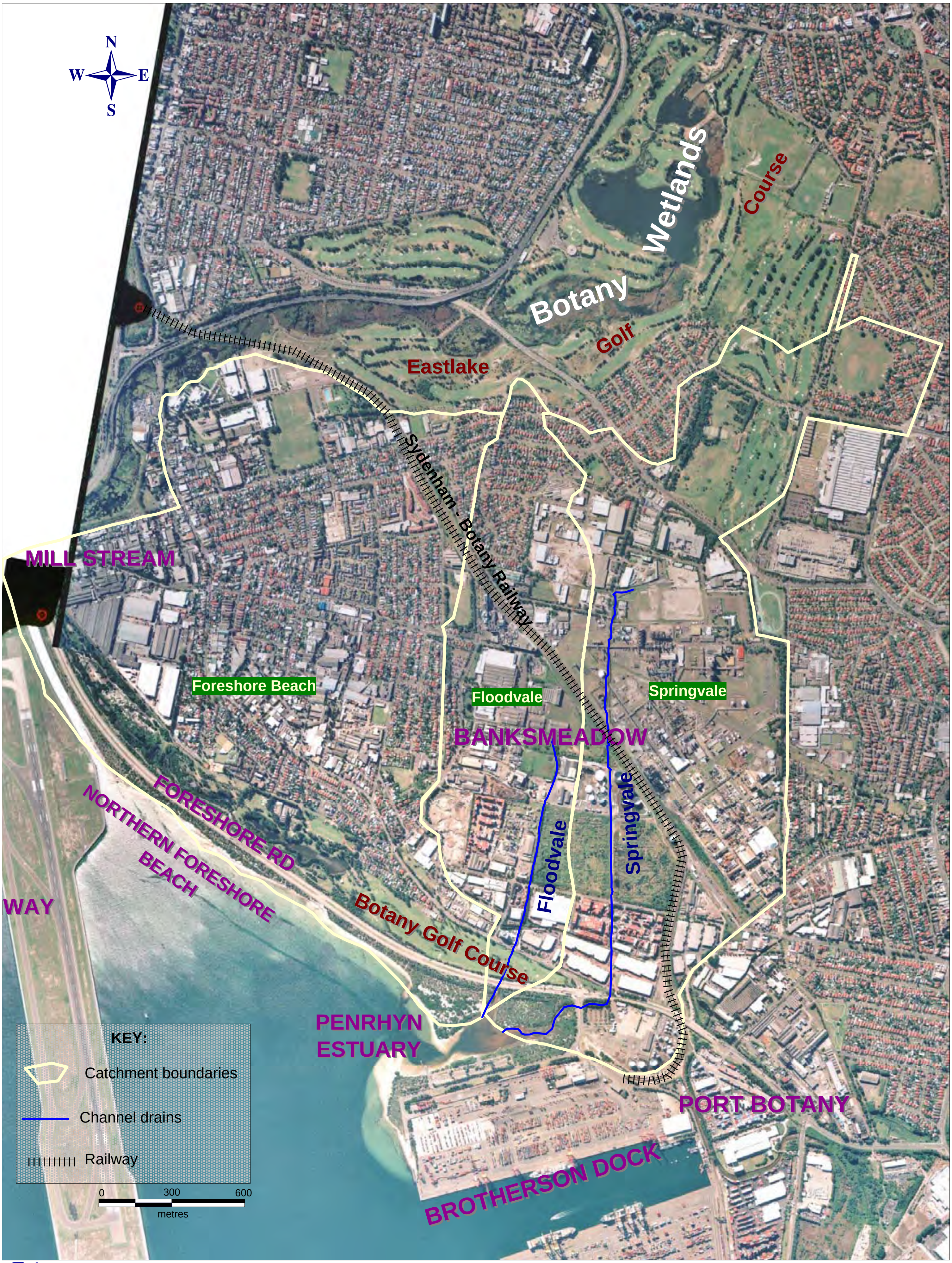
9. QUALIFICATIONS

This report has been prepared by Lawson and Treloar to assess flood impacts of the proposed port expansion for Sydney Ports Corporation (SPC). As such, this report is specific to this purpose and may not be used by third parties.

The investigation and modelling procedures adopted for this study follow current best practice and considerable care has been applied to the preparation of the results. However, model setup and validation depends on the quality of data available and there are levels of uncertainty for different types of data inputs. The hydrological and hydraulic regimes in the study area are complex and are represented by schematised model layouts.

Nonetheless, for the purposes of overall comparison of the pre and post development conditions, the models are adequate for the purpose of this study.

FIGURES

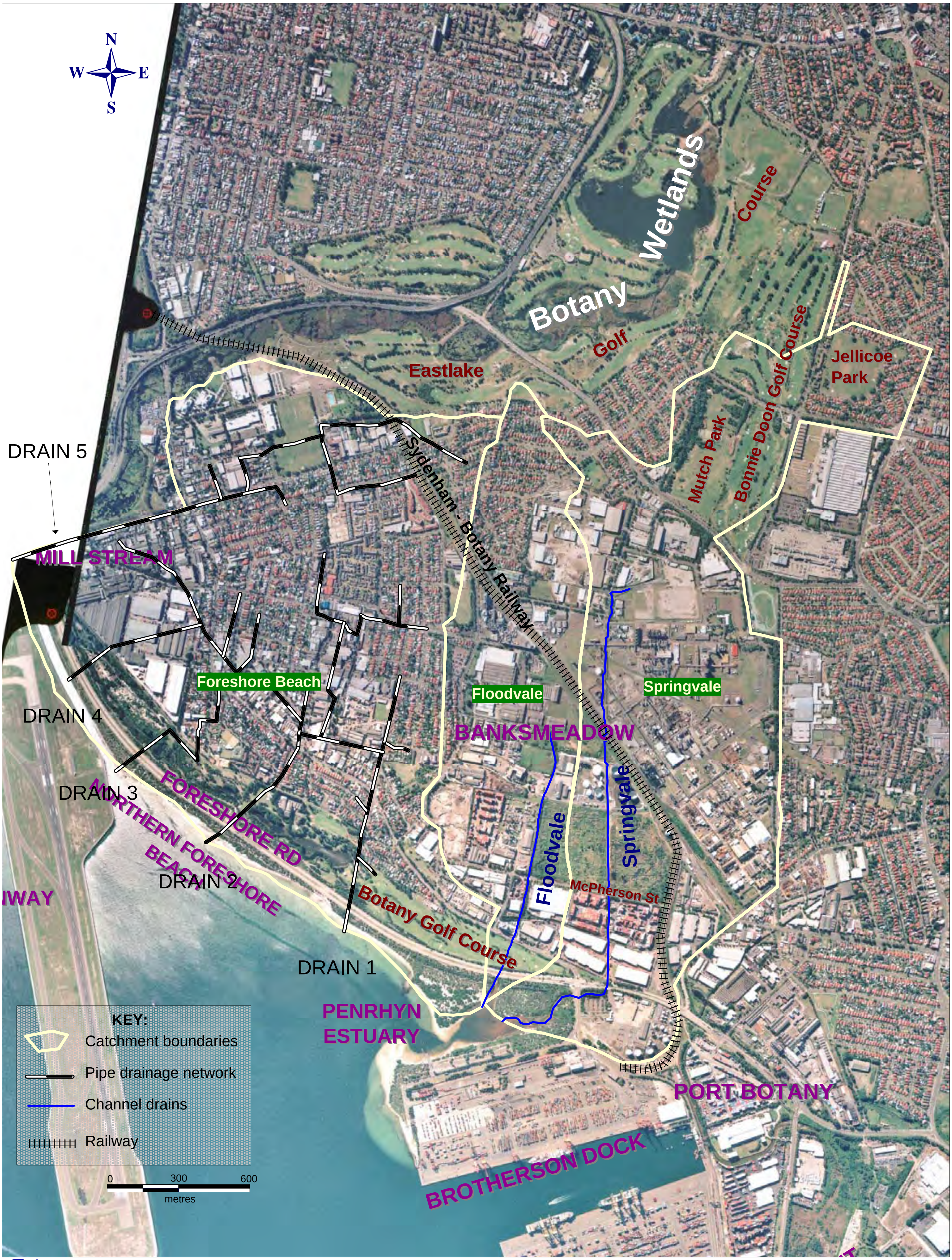


LAWSON & TRELOAR
J2076/R1999/ Volume 1/ Version 5
23/5/03

Proposed Expansion of Container Port Facilities, Botany Bay

FIGURE 1.1
CATCHMENTS LOCALITY PLAN

Era/Jobs/J2076/Vol1 Figures V5/Fig 1.1 catch.Wor



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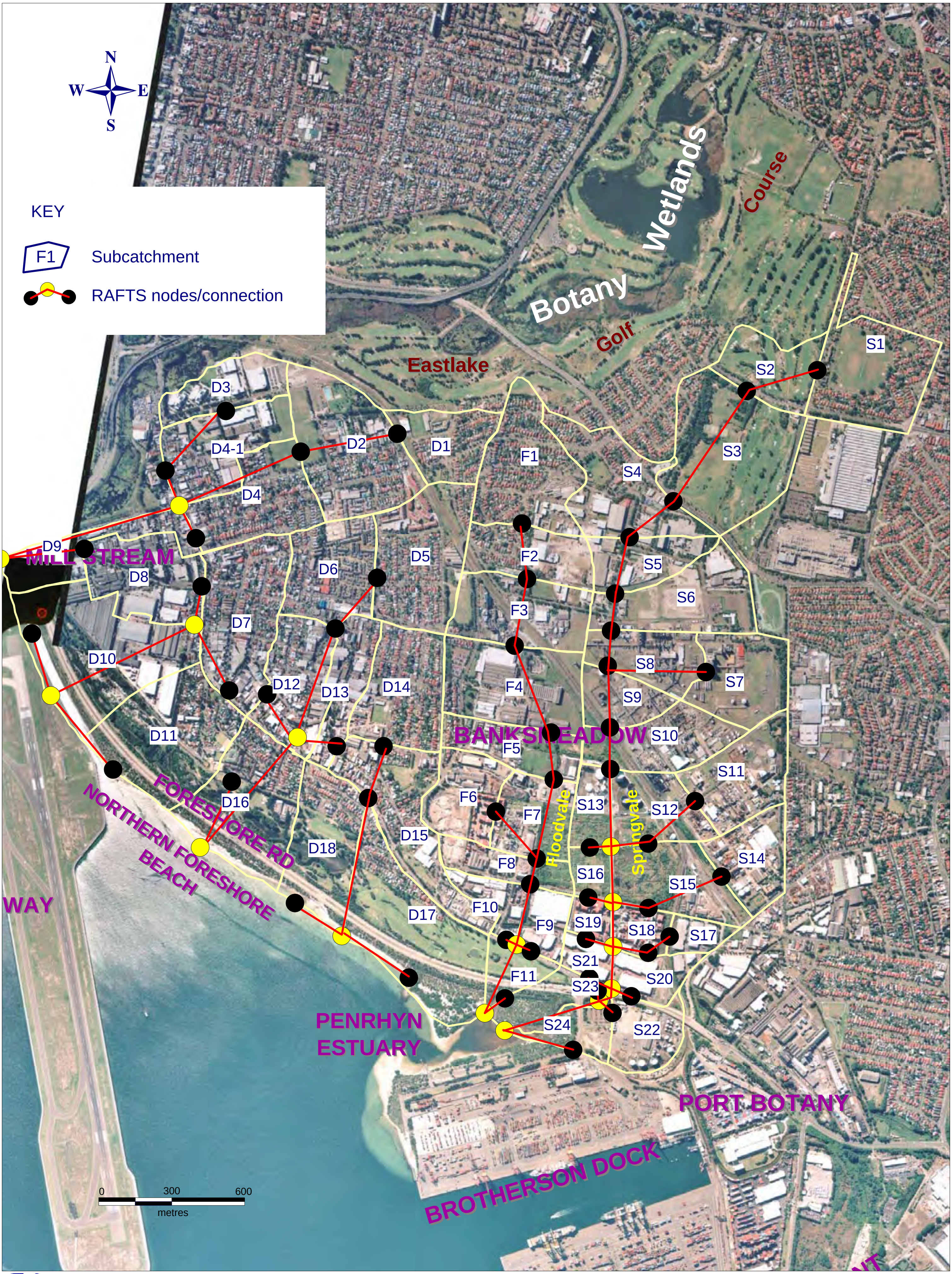
J2076/R1999/ Volume 1/ Version 5

23/5/03

Proposed Expansion of Container Port Facilities, Botany Bay

FIGURE 2.1
DRAINAGE PLAN

Era/Jobs/J2076/Vol1 Figures V5/Fig 2.1 drain.Wor



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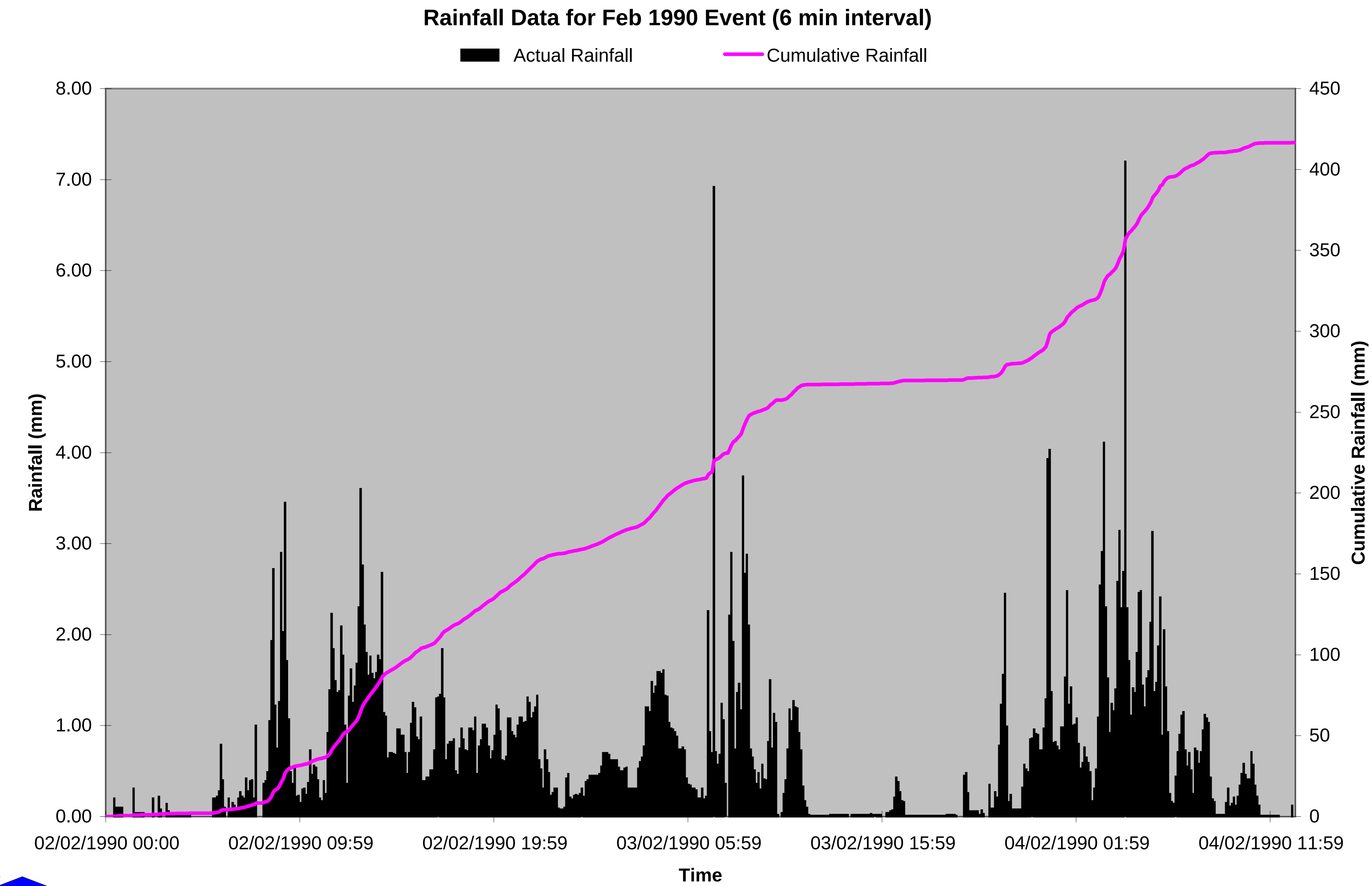
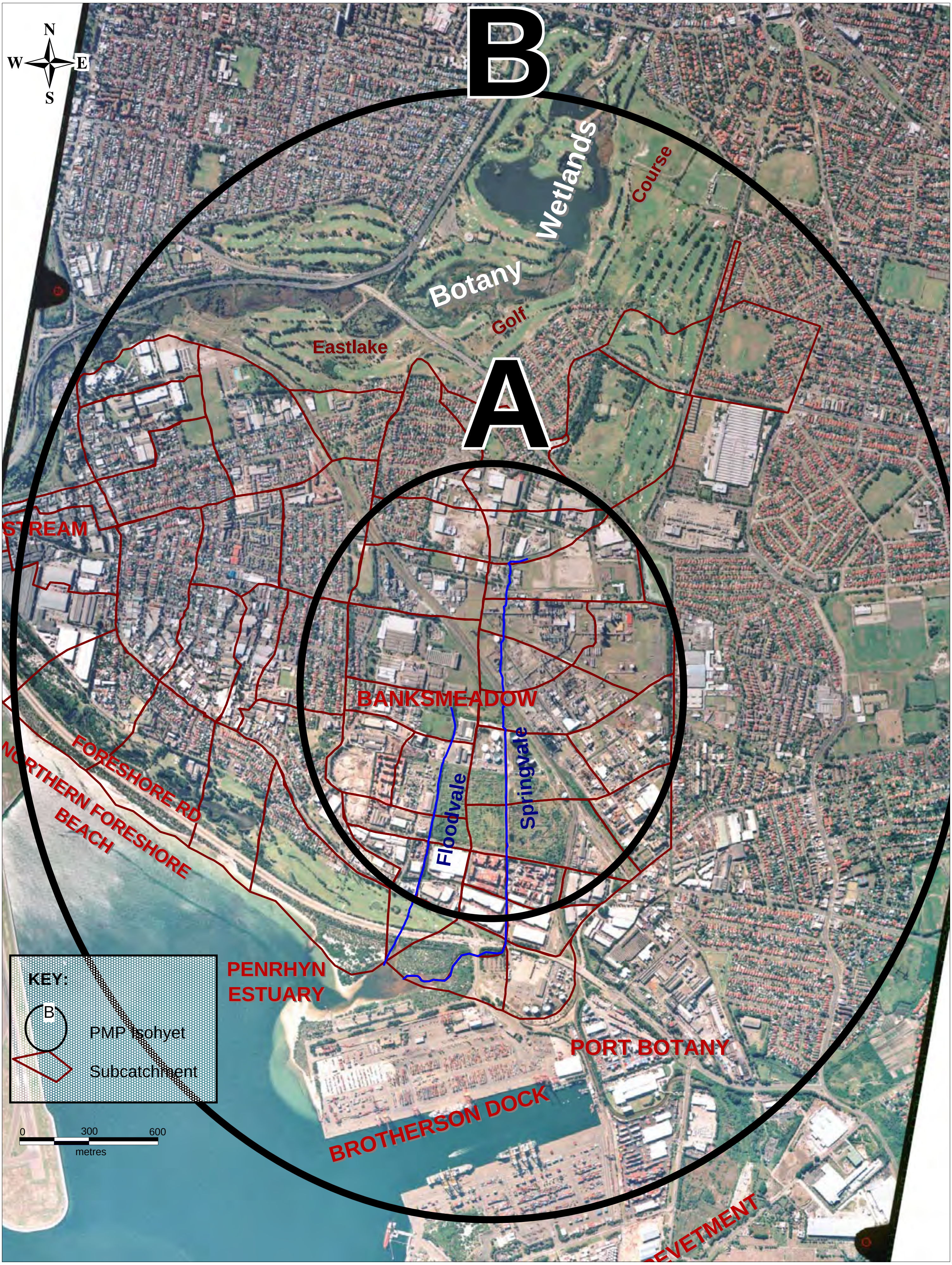
23/5/03

Proposed Expansion of Container
Port Facilities, Botany Bay

FIGURE 3.1
RAFTS MODEL LAYOUT

Era/d/Jobs/J2076/Vol1 Figures V5/Fig 3.1 subcatch.Wor


Rainfall Data for Feb 1990 Event (6 min interval)
Actual Rainfall
Cumulative Rainfall
Rainfall (mm)
Cumulative Rainfall (mm)
Time
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
450
400
350
300
250
200
150
100
50
0
02/02/1990 00:00
02/02/1990 09:59
02/02/1990 19:59
03/02/1990 05:59
03/02/1990 15:59
04/02/1990 01:59
04/02/1990 11:59



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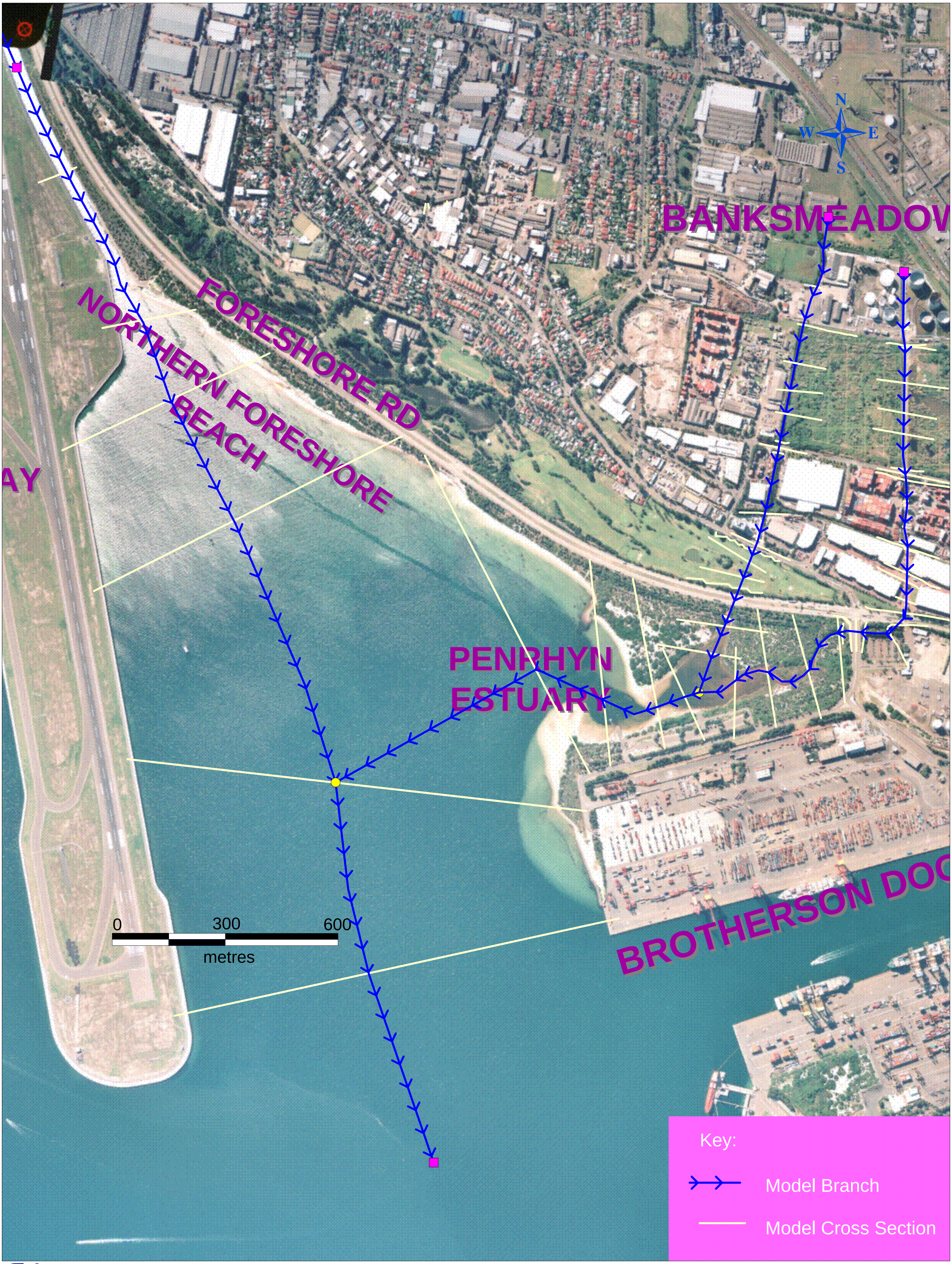
23/5/03

Proposed Expansion of Container
Port Facilities, Botany Bay

FIGURE 3.3
SPATIAL DISTRIBUTION OF PMP
Era/d/Jobs/J2076/Vol1 Figures V5/Fig 3.3 pmp.Wor



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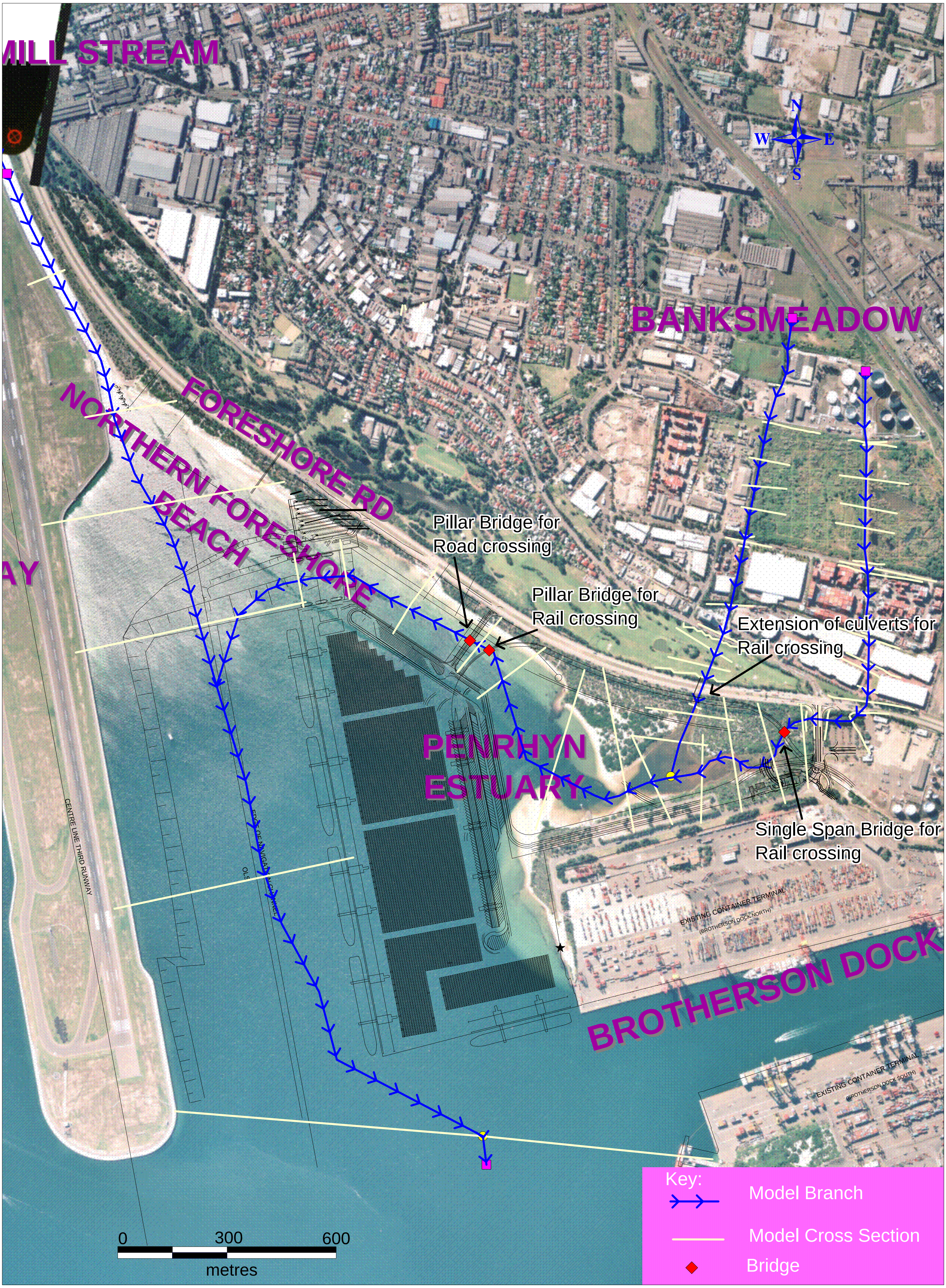
23/5/03

Proposed Expansion of Container
Port Facilities, Botany Bay

FIGURE 4.1
HYDRAULIC MODEL LAYOUT
EXISTING SCENARIO

Era/d/Jobs/J2076/Vol1 Figures V5/Fig 4.1 hydraulic layout.Wor



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J2076/R1999/ Volume 1/ Version 5
23/5/03

Proposed Expansion of Container
Port Facilities, Botany Bay

FIGURE 4.2
HYDRAULIC MODEL LAYOUT
PROPOSED PORT DEVELOPMENT SCENARIO
Era/d/Jobs/J2076/Vol1 Figures V5/Fig 4.2 hydraulic layout dev.Wor

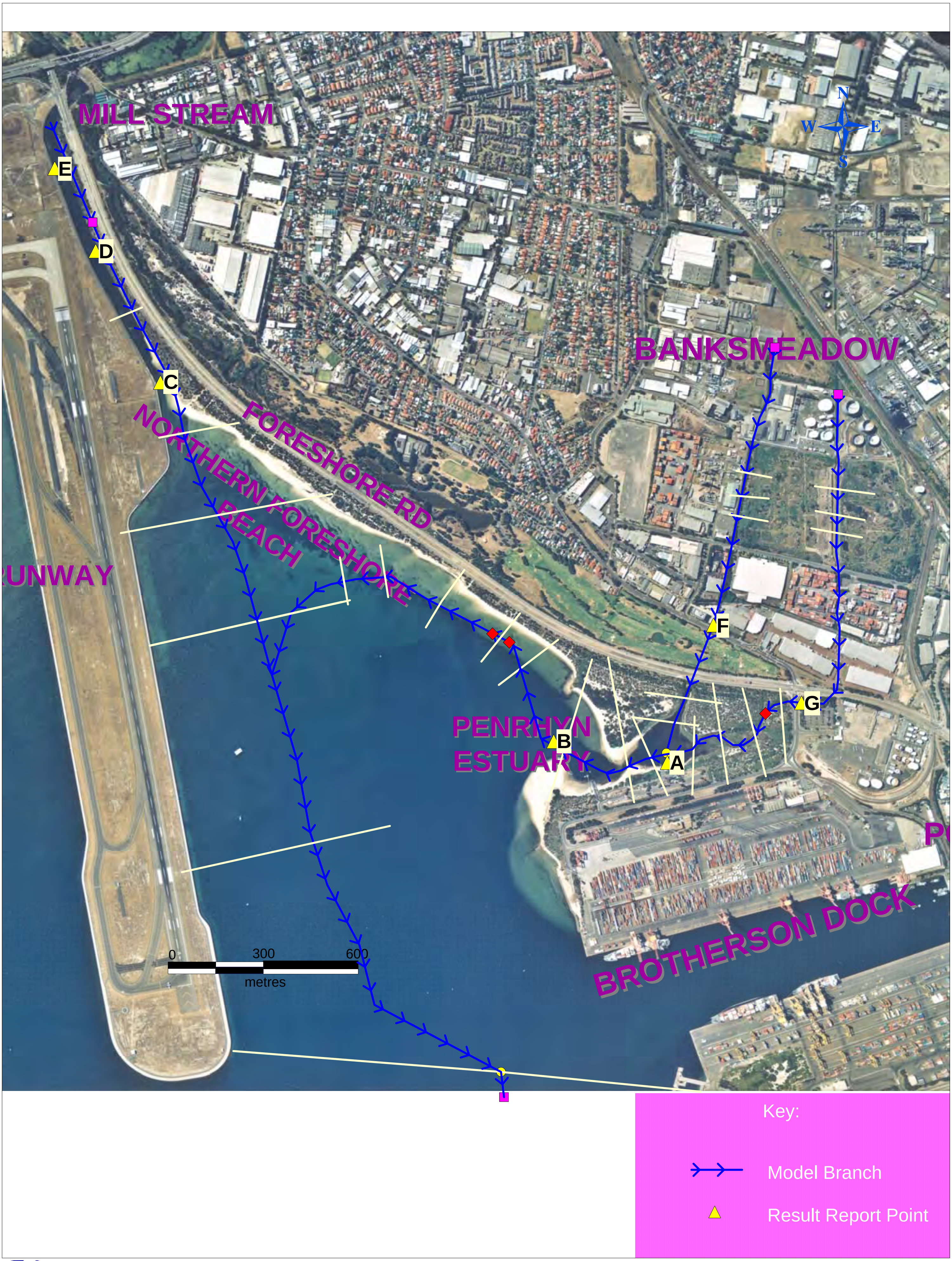

FIGURE 4.3
RESULT REPORTING LOCATIONS

APPENDICES

APPENDIX A

PROPOSED CONTAINER PORT EXPANSION DETAILS

-16	DENOTES MINIMUM DREDGED DEPTH
	BERTHING BOX (50m WIDE) TO BE DREDGED TO -16
	AREA TO BE DREDGED TO MAXIMUM DEPTH OF -17.5
	HIGH SPOTS TO BE DREDGED TO -16.5

Lawson and Treloar

J2076/R1999 Vol 3 18/02/2003

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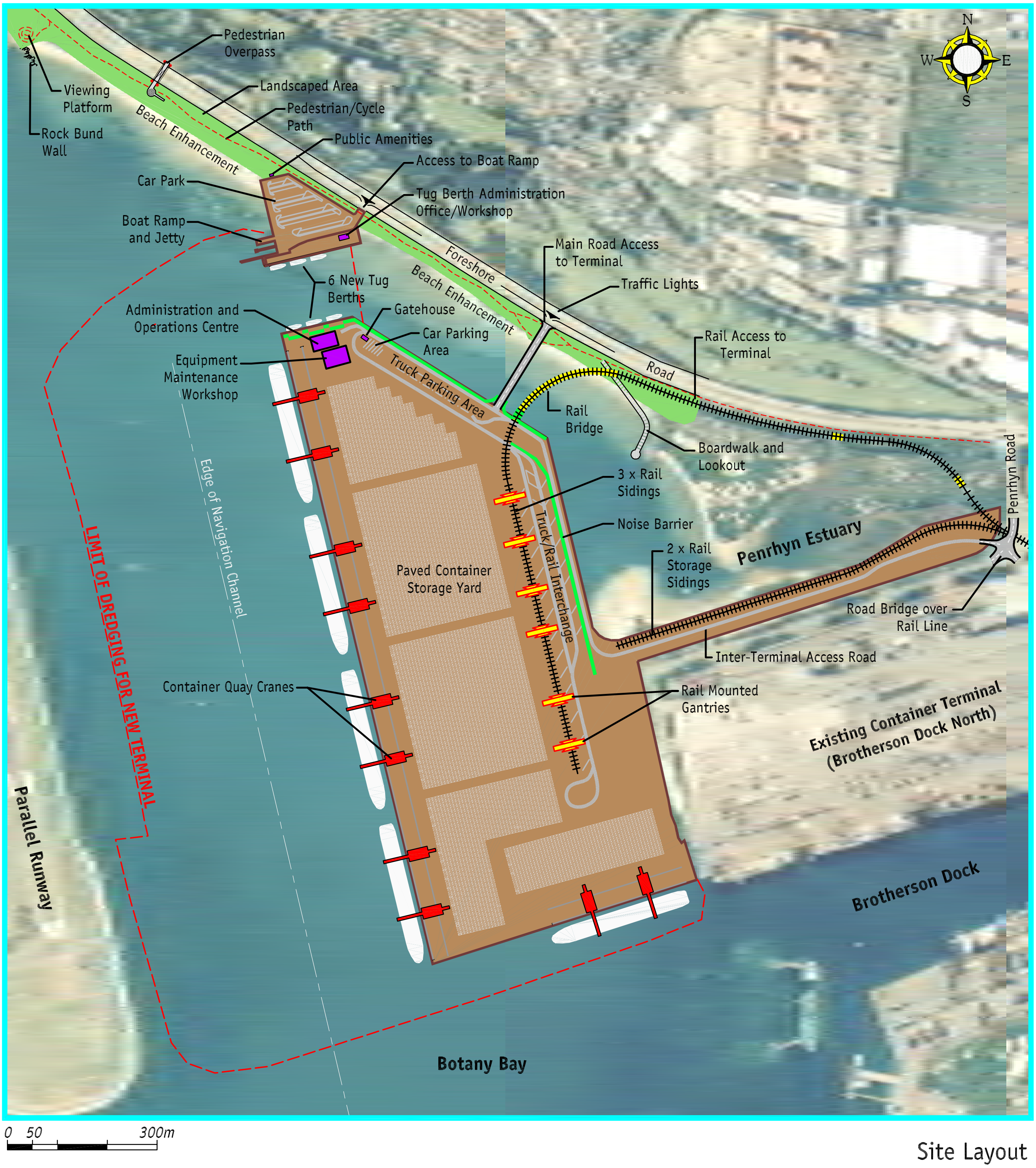
Proposed Expansion of Container Port Facilities, Botany Bay

PROPOSED BATHYMETRY OF PORT AREA

Appendix A

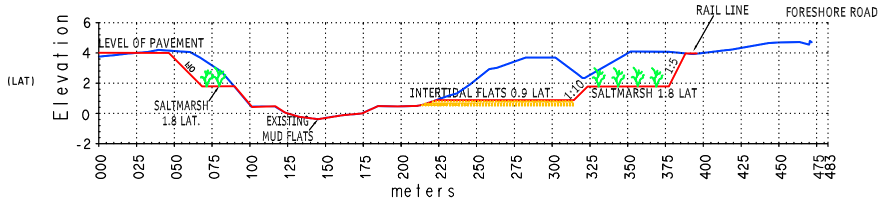


N
W
E
S
Pedestrian Overpass
Viewing Platform
Rock Bund Wall
Landscaped Area
Pedestrian/Cycle Path
Beach Enhancement
Public Amenities
Access to Boat Ramp
Car Park
Tug Berth Administration Office/Workshop
Boat Ramp and Jetty
Foreshore
Main Road Access to Terminal
6 New Tug Berths
Beach Enhancement
Traffic Lights
Administration and Operations Centre
Gatehouse
Car Parking Area
Rail Access to Terminal
Equipment Maintenance Workshop
Road
Truck Parking Area
Rail Bridge
Boardwalk and Lookout
Penrhyn Road
3 x Rail Sidings
Noise Barrier
Edge of Navigation Channel
Penrhyn Estuary
2 x Rail Storage Sidings
Paved Container Storage Yard
Truck/Rail Interchange
Road Bridge over Rail Line
Inter-Terminal Access Road
LIMIT OF DREDGING FOR NEW TERMINAL
Container Quay Cranes
Rail Mounted Gantries
Existing Container Terminal (Brotherson Dock North)
Parallel Runway
Brotherson Dock
Botany Bay
0 50 300m

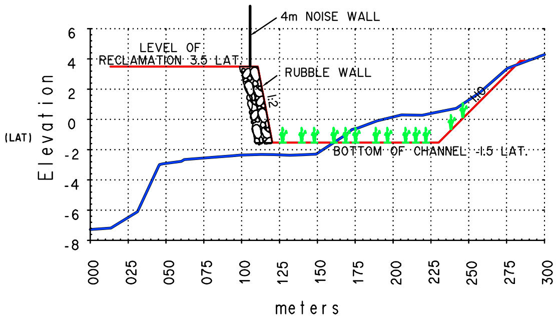
Site Layout

Penrhyn Estuary Proposed Habitat Enhancement Plan

- Proposed Intertidal Sand/Mud Flats (area 11.0ha)
- Existing Mudflats To Be Retained (area 1.5ha)
- Proposed Saltmarsh Habitat (area 5.2ha including 0.6ha of existing mangroves to be removed)
- Existing Saltmarsh To Be Transplanted into Proposed Saltmarsh Habitat (area 0.35ha)
- Existing Saltmarsh To Be Retained (area 1.0ha)
- Existing Mangroves To Be Removed & Replaced With Saltmarsh Habitat
- Proposed Seagrass Habitat (area 8.1ha)
- Potential Opportunity For Sediment/litter Traps (subject to detailed assessment on drain hydraulics)
- Proposed Preferential Flow Channel
- Proposed Preferred Noise Wall Location (approx. 4m High)



LINE 1



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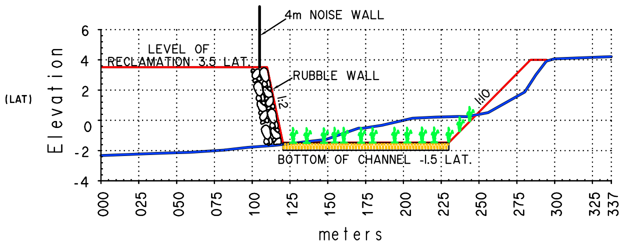

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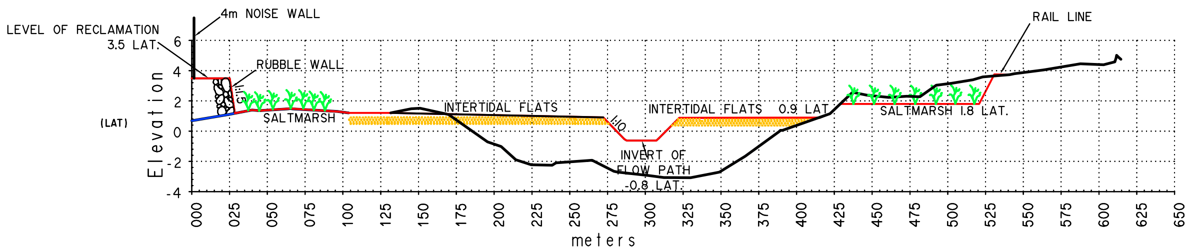

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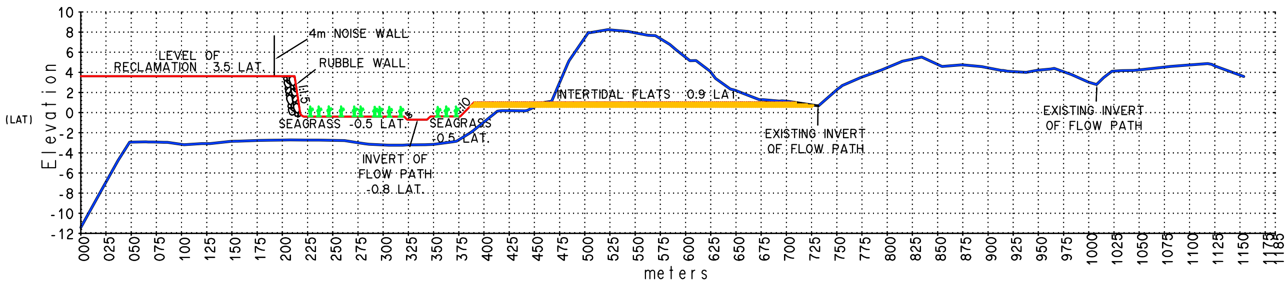

LINE 5

Vertical Exaggeration = 10x

All areas and measurements are approximations

Natural Surface

Proposed Development

Penrhyn Estuary Proposed Habitat Enhancement Cross Sections

APPENDIX B

GLOSSARY

GLOSSARY OF TERMS*

Annual Exceedence Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Recurrence Interval	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.
Cadastre, cadastral base	Information in map or digital form showing the extent and usage of land, including streets, lot boundaries, water courses etc.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design flood	A significant event to be considered in the design process; various works within the floodplain may have different design events. e.g. some roads may be designed to be overtopped in the 1 in 1 year or 100%AEP flood event.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flash flooding	Flooding which is sudden and often unexpected because it is caused by sudden local heavy rainfall or rainfall in another area. Often defined as flooding which occurs within 6 hours of the rain which causes it.
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 overland runoff before entering a watercourse and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.
Flood fringe	The remaining area of flood-prone land after floodway and flood storage areas have been defined.
Flood hazard	Potential risk to life and limb caused by flooding.
Flood-prone land	Land susceptible to inundation by the probable maximum flood (PMF) event, i.e. The maximum extent of flood liable land. Floodplain Risk Management Plans encompass all flood-prone land, rather than being restricted to land subject to designated flood events.

Floodplain	Area of land which is subject to inundation by floods up to the probable maximum flood event, i.e. flood prone land.
Floodplain management measures	The full range of techniques available to floodplain managers.
Floodplain management options	The measures which might be feasible for the management of a particular area.
Flood planning area	The area of land below the flood planning level and thus subject to flood related development controls.
Flood planning levels	Flood levels selected for planning purposes, as determined in floodplain management studies and incorporated in floodplain management plans. Selection should be based on an understanding of the full range of flood behaviour and the associated flood risk. It should also take into account the social, economic and ecological consequences associated with floods of different severities. Different FPLs may be appropriate for different categories of land use and for different flood plains. The concept of FPLs supersedes the "Standard flood event" of the first edition of the Manual. As FPLs do not necessarily extend to the limits of flood prone land (as defined by the probable maximum flood), floodplain management plans may apply to flood prone land beyond the defined FPLs.
Flood storages	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.
Floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often, but not always, aligned with naturally defined channels. Floodways are areas which, even if only partially blocked, would cause a significant redistribution of flood flow, or significant increase in flood levels. Floodways are often, but not necessarily, areas of deeper flow or areas where higher velocities occur. As for flood storage areas, the extent and behaviour of floodways may change with flood severity. Areas that are benign for small floods may cater for much greater and more hazardous flows during larger floods. Hence, it is necessary to investigate a range of flood sizes before adopting a design flood event to define floodway areas.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.
High hazard	Possible danger to life and limb; evacuation by trucks difficult; able-bodied adults would have difficulty wading to safety; potential for significant structural damage to buildings.
Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.

Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
Low hazard	Should it be necessary, people and their possessions could be evacuated by trucks; able-bodied adults would have little difficulty wading to safety.
Mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of the principal watercourses in a catchment. Mainstream flooding generally excludes watercourses constructed with pipes or artificial channels considered as stormwater channels.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff, pipe and overland stream flow.
Peak discharge	The maximum discharge occurring during a flood event.
Probable maximum flood	The flood calculated to be the maximum that is likely to occur.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedence Probability.
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. For this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
Stage hydrograph	A graph that shows how the water level changes with time. It must be referenced to a particular location and datum.
Stormwater flooding	Inundation by local runoff. Stormwater flooding can be caused by local runoff exceeding the capacity of an urban stormwater drainage system or by the backwater effects of mainstream flooding causing the urban stormwater drainage system to overflow.
Topography	A surface which defines the ground level of a chosen area.

* Many terms in this Glossary have been derived or adapted from the NSW Government *Floodplain Management Manual*, 2001.

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