Cardno MBK

Engineering the Future

VOLUME 1 – MAIN REPORT

LOWLANDS LAGOONS CATCHMENT DRAINAGE STUDY

HERVEY BAY CITY COUNCIL

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LOWLANDS LAGOONS CATCHMENT DRAINAGE STUDY

FINAL REPORT

VOLUME 1- MAIN REPORT

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STATUS	PRELIMINARY	☐
	DRAFT	☐
	DRAFT FINAL	☐
	FINAL	☑
	SUPERSEDED	☐
	OTHER (Specify)	

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1.0 INTRODUCTION

1.1 Overview

The Lowlands Lagoons and Tooan Tooan Creek catchments, with an area of 602 hectares, are situated on the coast of Hervey Bay between Pialba in the West and Urangan in the East. The catchment definitions applied by Hervey Bay City Council to the study areas are as follows:

- **C8.0 Lowlands Lagoons**
- **C9.0 Tooan Tooan Creek, containing the following subcatchments:**
  - **C9.1 Freshwater Street**
  - C9.2 Stephenson Street
  - C9.3 Taylor Street
  - C9.4 Neils Street
  - **C9.5 Confluence**

As subcatchments C9.2 (Stephenson Street) and C9.3 (Taylor Street) were studied in detail as part of other commissions, detailed modelling of the study area was limited to catchment C8.0 (Lowlands Lagoons) and subcatchments C9.1 (Freshwater Street) and C9.5 (Confluence).

The boundary of the study area is shown on **Figures 1A and 1B- Study Area**.

The drainage characteristics of the Eastern (Lowlands Lagoons) half of the catchment are dominated by a series of large interconnected lagoons. Due to the presence of a relatively high level frontal dune runoff from the catchment, other than that portion which ponds in interdunal areas, is initially directed inland to the Lowlands Lagoons. During flood events, the lagoons act as detention basins, temporarily storing runoff and attenuating peak flows and flood levels.

Runoff collected in the lagoons is ultimately discharged via:

- Stormwater drainage pipes which pass beneath the frontal dune and convey flow from the lagoons to Hervey Bay,
- Infiltration to groundwater, and
- Tooan Tooan Creek to Hervey Bay if levels are sufficiently high in the lagoons.

Of the stormwater drainage pipes that link the lagoons to Hervey Bay, the systems at Margaret Street and Churchill Street are the most significant. Designed to increase the rate at which the lagoons drain, both systems are fitted with flapgates to prevent the penetration of saline water to the lagoons during high tides and storm surge events. The use of the tide gates and inlet weirs allows a nominal standing water level of RL 1.5 m AHD to be maintained in the lagoons (refer Section 3.8.3). It can be noted that the nominal standing water level at Churchill Street is RL 1.5 m AHD. At present, a layer of bricks has been temporarily installed at the inlet to the Churchill Street system to raise the level in the upstream lagoon to RL 1.7 m AHD.

The Western portion of the catchment also features a frontal dune system. Runoff from this part of the catchment drains to Tooan Tooan Creek, which outlets to Hervey Bay at Pialba. Tooan Tooan Creek also receives runoff from the Stephenson Street and Pialba catchments. These catchments discharge to the creek near its mouth and therefore exert only a relatively minor influence upon flood levels in the creek.

During flood events, the initial runoff from part of the western catchment enters the creek and travels in an easterly rather than westerly direction. This water ultimately enters and is stored in the lagoons. In the later stages of flood events, the level in the lagoons increases sufficiently to cause the lagoons to discharge to the creek. An existing slight high point in the creek between Denmans Camp Road and Tavistock Street governs the distribution of flow to and from the lagoons.

For the majority of its length, Tooan Tooan Creek takes the form of an engineered concrete lined channel. In the region between Frank Street and Queens Road, the creek is piped via a number of box culverts for a length of about 200 metres. The channel is relatively narrow given the catchment area it commands. The channel also features numerous driveway and road crossings that reduce its hydraulic efficiency even further. The capacity of the channel is so small in some reaches that it is considered the only reason the channel does not flood for even the most minor rainfall events is the storage and flow attenuation provided by the lagoon system.

In the region to the north of the creek and lagoons, existing ground levels vary from as low as RL 2.0 m AHD to over RL 4.0 m AHD, with an average level of about RL 3.5 m AHD. In comparison, the level of Highest Astronomic Tide is RL 2.1 m AHD and the tailwater level nominated for the consideration of major events is RL 2.4 m AHD (Hervey Bay City Council 1997, p 16-14). As a consequence of these levels, properties within the Lowlands catchment are regularly inundated during storm events.

Ground levels increase to the south of the creek and lagoon system, with maximum levels of up to RL 26 m AHD occurring. As a consequence of these relatively high levels and the completion of flood mitigation works in identified problem areas (GHD 1996), flooding in areas to the south of the creek and lagoon system is generally not as significant as that experienced to the north of the creek.

Present flooding problems are exacerbated by four factors:

- Lack of capacity in Tooan Tooan Creek;
- Limited drainage of Lagoon system;
- Lack of maximisation of storage potential of lagoons; and
- Undersized stormwater drainage system in most parts of the catchment.

Of the above factors, the undersized stormwater drainage system would be expected to produce localised flooding resulting from water ponding at gully pits in upstream areas. Although undesirable, the present ponding of water would be reducing the peak flow in the creek and thereby minimising flooding in the main creek.

Cardno MBK was commissioned by Council to determine the most cost effective and environmentally acceptable means for improving the drainage of the catchment. As noted above, due to the interaction of the Lowlands Lagoons and Tooan Tooan Creek, the catchment needed to be analysed both dynamically and as a single catchment. For this reason, a model capable of dynamically modelling stormwater drainage and lagoon systems was selected (refer Section 1.2).

This report (Volume 1) details the findings of the study and recommends a range of relief drainage measures for the catchment (refer Section 5). Volume 2 of the report contains the appendices to the main report.

1.2 Modelling Package

The catchment was analysed using Version 8.0 of the XP Software XP-UDD package (XP Software 2000). XP-UDD replaces the formerly separate RAFTS runoff routing and EXTRAN hydraulic models.

XP-UDD comprises Runoff and Hydraulic (EXTRAN) modes to allow the simulation and hydraulic routing of rainfall. The Hydraulic (EXTRAN) mode of the package is a hydraulic flow routing model for both open channel and closed conduits in dendritic and looped networks. The Hydraulic mode receives hydrograph input at specific nodal locations directly from the output of the Runoff mode of the program. The model uses a combination of implicit and explicit finite difference formulations to dynamically route runoff throughout the modelled drainage system.

The XP-UDD model used for the analysis had the following capabilities:

- Maximum Nodes: 1,000
- Maximum Links: 1,000
- Maximum Texts: 40
- Maximum Pictures: 10
- Maximum Cards: 150,000

License options for the model included:

- Profile Plotting
- Pumps/ Orifices
- Full Equations (rather than simple kinematic wave approach for conduits)
- All conduit shapes
- DXF and AutoCAD DWG background picture input

A major feature of the program is that data can be input to XP-UDD models entirely via ASCII text files. Extensive use of this facility was made during the investigation to improve the quality of the model and to facilitate data entry. Data was input to EXCEL spreadsheets, output in a suitable text file format and then imported to the XP-UDD model. For the study, cross sectional data was stored in an ACCESS database.

2.0 HYDROLOGY

2.1 Overview

To correctly represent the runoff characteristics of the study area, the 602 hectare catchment was divided into a total of 261 subcatchments with an average area of 2.3 hectares. This fine level of discretisation was considered necessary to ensure that the runoff from the catchment could be modelled in sufficient detail. Runoff hydrographs were derived for the following development scenarios:

- Existing catchment (at the time of the Asset Data Capture program), and
- Ultimate catchment development, as defined in Development Control Plan 1 (refer **Figure A2- Zoning Plan**).

The adopted sub catchment layout is shown on **Figure A1- Catchment Plan** of **Appendix A- Hydrologic Data**. Data pertinent to each subcatchment are presented in the **Table A1- Catchment Areas, Weighted Runoff Coefficients and Times of Concentration, Existing development** and **Table A4- Catchment Areas, Weighted Runoff Coefficients and Times of Concentration- Ultimate Development** in Appendix A.

The division of the catchment was based upon internal ridge lines and existing inlets to the stormwater drainage system. In some cases, local sags were found within individual properties. In such cases, it was assumed that property owners would eventually fill their blocks to remove the sags and direct runoff to the road and stormwater drainage system. Each subcatchment area was assigned to a node in the XP-UDD model (refer **Figure A1- Catchment Plan**).

Hydrographs were derived for each subcatchment for storm durations ranging from fifteen minutes to thirty-six hours for events with recurrence intervals of 2 years (minor event) and 100 years (major event). The standard storm durations presented in *Australian Rainfall and Runoff* (Institution of Engineers 1987) were considered applicable to the catchment area. As the Lowlands catchment is highly urbanised, its subcatchments respond rapidly and produce their peak runoff for relatively short duration events. However, the storage available within and the flow attenuation afforded by the lagoons means that peak flood levels in the lagoon are governed by events with longer durations. For this reason, it was found necessary to model events with durations of up to 36 hours in order to determine the peak level reached in the lagoons.

In order to facilitate future modelling of intermediate development scenarios and to provide maximum consistency with the Queensland Urban Drainage Manual (QUDM) (Neville Jones & Associates et al 1992) and the Hervey Bay City Council Development Manual (Hervey Bay City Council 1997), simplistic runoff hydrographs were derived based on the Rational Method. As described in the following sections, this was considered to be acceptable given the number of subcatchments into which the catchment was divided.

The peak flows calculated for the 2 and 100 year events for each subcatchment are presented in the following tables in **Appendix A- Hydrologic Data**:

- **Table A2- Peak Flows, 2 Year Event, Existing Catchment**
- **Table A3- Peak Flows, 100 Year Event, Existing Catchment**
- **Table A5- Peak Flows, 2 Year Event, Ultimate Catchment**
- **Table A6- Peak Flows, 100 Year Event, Ultimate Catchment**

Because the Stephenson Street and Taylor Street catchments drain to Tooan Tooan Creek, it was necessary to add inflows to the model that accounted for the runoff from the two catchments. Given the size of the Stephenson Street and Pialba catchments (90 and 210 hectares respectively), it was not considered appropriate to use the method of hydrograph derivation adopted within the smaller Lowlands subcatchments.

As Council had developed XP-UDD models of both the Stephenson and Taylor Street catchments, it was possible to use the models to provide the necessary runoff hydrographs for use in the Lowlands model (refer Section 2.4).

2.2 Derivation Of Hydrographs For Lowlands Catchment

The Runoff mode of XP-UDD provides the following methods of hydrograph generation (XP-Software 1998, pp 164-174):

- SWMM Runoff Non-Linear Reservoir Method,
- Kinematic Wave Method,
- Laurenson Non-Linear Method (RAFTS),
- SCS Unit Hydrograph Method,
- Other Unit Hydrograph Methods including:
  - Nash,
  - Santa Barbara Urban Hydrograph,
  - Snyder,
  - Snyder (Alameda Modified),
  - Time Area (ILSAX), and
  - Rational Formula,
- User Defined Hydrograph.

An evaluation of the available hydrograph derivation techniques was undertaken for a recent study completed for Council (Cardno MBK 2000, Section 2.2). The evaluation determined that a Rational Method based approach to the derivation of hydrographs and the use of User Defined hydrographs for the importation of the hydrographs to the hydraulic model was optimal for the investigation. Consequently, a similar approach was adopted for this investigation.

In order to provide the desired hydrographs based on the Rational Method, use was made of the option available in XP-UDD to directly input "User Defined" hydrographs to the model (XP Software 1998, p 148). It can be noted that this technique differs from the other methods of hydrograph generation in that hydrographs are directly input to the model rather than being generated by the Runoff mode of the XP-UDD package. When the Runoff mode of the package is used, the program calculates hydrographs and then generates an interface file that is read at the start of the Hydraulic or EXTRAN mode run. The Runoff and Hydraulic modes of the program are otherwise entirely separate models.

The use of user-defined hydrographs obviated the need to use the Runoff mode of the package. Due to the relatively simplistic nature of the hydrographs input to the model, the hydrographs were produced using a spreadsheet. The spreadsheet was formatted in a manner that allowed the output of text files in a format that could be directly input to the model using the XPX file importation option of XP-UDD (XP Software 1998, pp 110-115).

The runoff hydrograph spreadsheet featured a master worksheet containing the following information for each subarea of the catchment:

- Name of nodes in the model to which inflow was to be attached,
- Time of concentration of the subarea
- Total area of the subcatchment
- Breakdown of area into the various zones present within the catchment (eg Business Development)
- Calculated runoff coefficients for the 2 and 100 year events weighted according to the composition of the subcatchment.

Underlying worksheets took the information presented in the master worksheet and provided tables of hydrographs in XPX file format. For the recurrence intervals considered, worksheets were produced for each storm duration requiring investigation. The only information input to the underlying worksheets was the rainfall intensity represented by the worksheet. A separate spreadsheet was prepared containing storm duration information in XPX format.

The contents of each worksheet were output as text files and then imported to the hydraulic mode XP-UDD model as required. The importation of a data file for each storm duration and recurrence interval under investigation was similar in terms of user effort to specifying the interface file which would have been produced if the Runoff mode of the package had been used to derive hydrographs.

In order to provide an acceptable coverage of the range of storm durations that could potentially cause peak flood levels and flows within the catchment, the standard storm durations presented in *Australian Rainfall and Runoff* (Institution of Engineers Australia 1987) were adopted for the investigation. It was considered that the time difference between the standard storm durations for the short duration (15 to 20 minute) storms likely to produce peak conditions for minor events was sufficiently small to negate the requirement for the assessment of intermediate durations.

In summary, it was found that the optimal method for hydrograph derivation given the nature of the catchment and the method of analysis being applied to the catchment was provided by a direct application of the Rational Method. This provided inflow hydrographs entirely consistent with the *Hervey Bay City Council Development Manual* (Hervey Bay City Council 1997) and the *Queensland Urban Drainage Manual*.

2.3 Derivation Of Hydrologic Parameters

2.3.1 Time of Concentration

Section 5.05.4 of the *Queensland Urban Drainage Manual* (QUDM)(Neville Jones & Associates 1992, p 5-15) notes that:

"The use of standard inlet times for developed catchments is recommended because of the uncertainty related to the calculation of time of overland flow. The standard inlet times should be adopted except where subcatchment characteristics indicate that detailed overland flow calculations are justified."

Section 16.3.3 of the *Hervey Bay City Council Development Manual* (Hervey Bay City Council 1997, p 16-6) allows the use of the standard inlet times presented in QUDM.

Given the above and the transparency of calculation offered by the use of standard inlet times, the standard inlet times presented in Table 5.05.1 of QUDM were adopted for the analysis. Due to the relatively low grades within the study area, the majority of the subareas were assigned a time of concentration of 15 minutes.

As the inlet time is defined as "the combined time for overland flow and channel flow to the gully inlet under consideration" (Neville Jones & Associates et al 1992, p 5-15), the time of concentration at nodes not directly connected to the underground stormwater drainage network was reduced to account for the likely channel (i.e. road) travel time to the closest inlet to the inflow point.

The time of concentration adopted for each subcatchment is shown in **Table A1- Catchment Areas, Weighted Runoff Coefficients and Times of Concentration** in **Appendix A- Hydrologic Data**.

2.3.2 Rainfall Intensity

Rainfall intensities for the catchment were derived in accordance with Volumes 1 and 2 of Australian Rainfall and Runoff (Institution of Engineers Australia 1987). The derived rainfall intensities are listed in **Table 2.1- Design Rainfall Intensities**.

TABLE 2.1
Design Rainfall Intensities

Storm Duration	Rainfall Intensity (mm/hour)	
	2 Year Event	100 Year Event
15 minutes	94.8	200
20 minutes	82.8	174
25 minutes	74.1	156
30 minutes	67.5	142
45 minutes	54.3	113
1 hour	46.2	96.3
90 minutes	35.7	75.9
2 hours	29.6	63.8
3 hours	22.7	49.9
4.5 hours	17.5	39.2
6 hours	14.3	32.7
9 hours	11.1	25.7
12 hours	9.08	21.5
18 hours	7.10	17.6
24 hours	5.95	15.3
30 hours	5.28	13.9
36 hours	4.60	12.4

2.3.3 Fraction Impervious Values and Runoff Coefficients

Fraction Impervious values for the existing catchment were defined based on the aerial photography collected for Council's Asset Data Capture program. The fraction impervious values were correlated with the runoff coefficients adopted for various landuses within Hervey Bay and suitable runoff coefficients for each subcatchment derived. The fraction impervious values and runoff coefficients adopted for each subcatchment for the existing case are listed in **Table A1- Catchment Areas, Weighted Runoff Coefficients and Times of Concentration, Existing development** in **Appendix A- Hydrologic Data**.

The existing catchment was found to have an average fraction impervious value of 0.36.

For the analysis of ultimate catchment development, appropriate fraction impervious values for each type of landuse within the catchment were derived based on the zonings presented in the Hervey Bay City Council Development Control Plan 1 (DCP1, November 1999). The zoning plan for the Lowlands catchment is reproduced as **Figure A2- Zoning Plan** in **Appendix A- Hydrologic Data**.

The predominant land use within the catchment is a mixture of residential cottage development and multiple unit development. The 602 hectare catchment contains the following land use types:

- Residential Cottage Development (coloured pink on DCP1) 360 ha
- Multiple Unit Development (coloured red on DCP1) 155 ha
- Business Unit Development (coloured blue on DCP1) 23 ha
- Open Space (no colour on DCP1) 64 ha

It can be noted that the above areas include the areas of road adjacent to each land use.

The area of each type of zoning present within the subcatchments defined for the investigation is presented in **Table A4- Catchment Areas, Weighted Runoff Coefficients, and Times of Concentration- Ultimate Development** in **Appendix A- Hydrologic Data**. The ultimate catchment was found to have an average fraction impervious value of 0.64.

The fraction impervious values listed for various development categories in the *Queensland Urban Drainage Manual* (Neville Jones & Associates 1992, p 5-10) were correlated with the various land use zonings present within the study area according to the *Hervey Bay City Council Development Manual* (Hervey Bay City Council 1997, p 16-6) to provide appropriate fraction impervious values for each land use within the catchment.

The adopted fraction impervious values were then applied to Table 5.04.2 of QUDM (Neville Jones & Associates et al 1992, p 5-11) to derive suitable values of the runoff coefficient for the 10 year event. The derived values were subsequently compared to those adopted for the *Toogoom Area Drainage Study* (John Wilson and Partners 1998) and any coefficients significantly different to those adopted for the Toogoom study altered appropriately.

Frequency factors of 0.85 and 1.20 were applied to the 10 year event coefficients to derive the coefficients applicable to the 2 and 100 year events respectively (Neville Jones & Associates 1992, p 5-11). As recommended by QUDM, runoff coefficient values in excess of unity were rounded down to 1.0.

The adopted values of fraction impervious and runoff coefficient are listed in **Table 2.2- Adopted Fraction Impervious Values and Runoff Coefficients**.

TABLE 2.2
Adopted Fraction Impervious Values and Runoff Coefficients

Land Use	QUDM Equivalent	Adopted Fraction Impervious	Adopted Runoff Coefficient		
			2 Year	10 Year	100 Year
Residential Cottage Development	Urban Residential High Density	0.7	0.701	0.825	0.990
Multiple Unit Development	Urban Residential High Density	0.8	0.723	0.850	1.000
Business Development	Central Business	1.0	0.765	0.900	1.000
Open Space	Open Space	0.0	0.561	0.660	0.792

2.4 Modelling of Stephenson Street and Taylor Street Catchments

As noted in Section 2.1, the Stephenson Street and Taylor Street catchments discharge to Tooan Tooan Creek near the mouth of the creek. As a consequence of this, it was necessary to model the runoff from these catchments in order to correctly represent flow conditions at the creek mouth.

It was not considered appropriate to use rational method hydrographs to represent runoff from the catchments due to their relatively large size and the presence of retention basins within the catchment (for ultimate conditions).

As Council had already modelled the two catchments previously using XP-UDD, the models were used to provide the necessary runoff hydrographs for the Lowlands catchment. Runoff hydrographs were calculated for storm durations of between 15 minutes and 36 hours for the 2 and 100 year events.

The calculated hydrograph for the Stephenson Street catchment was input to the model at node SNSTE02, while the calculated hydrograph for the Taylor Street catchment was input to the model at node SNSTE01.

2.5 Diversion of Runoff From Urangan Catchment

The drainage solution adopted for a site (known colloquially as the Caltex site) at the upstream end of the adjacent Urangan catchment is to drain runoff from the site to the lagoon system (refer Section 4.5.5 and **Figure 24- Adopted Relief Drainage Works**). This allows the size of drainage infrastructure required for the adjacent catchment to be reduced.

The runoff hydrographs prepared for the catchment were revised to include the runoff from a total area of 5.26 hectares. Of this area, 4.43 hectares was assumed to drain directly to the Kondari Resort lagoon (node SNLOWS02), with the remaining 0.83 hectares draining to Dayman Street (node SNLOWSSE02).

The revised data for the subcatchments draining to node SNLOWS02 and node SNLOWSSE02 are presented in **Table A7- Catchment Areas, Weighted Runoff Coefficients and Times of Concentration, Ultimate Development- Including Diversion from Urangan Catchment** in **Appendix A- Hydrologic Data**. Peak flow rates for the nodes are presented in **Table A8- Peak Flows, 2 and 100 Year Events, Ultimate Catchment- Including Diversion from Urangan Catchment** in Appendix A.

3.0 HYDRAULIC MODELLING

3.1 General

The XP-UDD model was used to develop hydraulic models of the Lowlands catchment representing existing (at the time of the Asset Data Capture Program) and ultimate levels of development. Section 2.1 and Section 3.2 of the report detail the changes associated with ultimate catchment development. The ultimate case model was subsequently modified to allow a number of relief drainage scenarios to be analysed (refer Section 4). Based on the results obtained from the analysis of available relief drainage scenarios, a set of relief drainage options was modelled (refer Section 4.7).

Table 3.1- XP-UDD Model Size lists the number of links and nodes used in the existing, ultimate, and relief drainage case models. In order to allow relatively straightforward use of the model in the future, the maximum number of links and nodes in the model was limited to 1,000.

TABLE 3.1
XP-UDD Model Size

Item	Existing Model	Ultimate Model	Combined Relief Drainage
Links	952	964	963
Outfalls	19	17	14
Weirs	3	4	4
Natural Channels	404	404	404
Storages	13	13	14

The XP-UDD model treats outlets as additional links for the purposes of modelling. Similarly, multilinks are treated as having a number of links equal to the number of culverts and weirs defined for the multilink. For example, a road crossing with a culvert beneath would be modelled as a culvert and weir and would be treated as two links by the model.

As a consequence, the total number of links modelled in each run was 952 for the existing case, 964 for the ultimate case, and 963 for the combined relief drainage case compared to the maximum limit of 1,000. Although it had been originally intended to use a lesser number of links for the existing case model, there are still a number of spare links available for the consideration of additional flood mitigation scenarios.

Due to the size of the catchment being modelled, it was not possible to construct a model to the level of detail that would be the most desirable. Consequently, when forming the model, compromises in model detail were necessary. Preference was given to modelling the main flow paths and the low-lying region to the north of Tooan Tooan Creek and the lagoons where the majority of flooding problems would be expected to occur. Initial model schematisations produced link numbers well in excess of the maximum allowable number of 1,000. Subsequent refinements to the model involved the modelling in lesser detail of areas to the south of Tooan Tooan Creek. Ultimately, a model schematisation with less than 1,000 nodes was achieved.

It can be noted that the modelling assumed that private properties would generally have little or no flow capacity. The existing road network was assumed to convey overland flow within the catchment. Although this approach meant that the storage available in such areas was not accounted for, it would not have been possible to include such flow paths and produce a model with less than 1,000 nodes. It was recognized that this would result in the conservative overestimation of peak flood levels for the existing case.

However, as it is likely that filling will occur on these properties over time, any assumed flow paths may not exist in the future. In comparison, the road network has a high hydraulic capacity compared to that of overland flow paths through overland flow points and will always be available to convey flow.

Other simplifications included not modelling the side drainage of stormwater lines where the head loss in the side drainage was not likely to be an issue.

To facilitate the use of the model, the following colour convention was adopted:

- Surface links: Red
- Surface Nodes: Red
- Underground Links: Blue
- Underground Nodes: Blue
- Surface to Underground Links: Black

As the number of characters which can be used to name links and nodes is limited to 10 characters, the following naming convention was adopted:

Surface Node name: SNAAAAA%%
Underground Node Name: UNAAAAA%%

Where SN, UN	=	Surface node and underground node respectively
AAAAA	=	Abbreviation of location of node.
%%	=	Drainage Line number (e.g. 01, 02 etc for each catchment)

Upper case letters were used to define all node names.

In the case of the nodes located within streets, the first four letters of the street name were generally adopted, with an N, C, or S then used to signify if the node was located to the north of the Lowlands Lagoons, between two of the lagoons, or to the south of the lagoons respectively.

The street names adopted for the naming of nodes were those that ran in a north south direction. This was because a number of the streets that run in an east-west direction extend throughout the catchment (e.g. Charlton Esplanade) and it would therefore be difficult to locate nodes named after such roads. In the case of nodes located in these streets, the node was assigned a name based on the names of the north-south streets on either side of the street. For instance, a node located between Alexander and Margaret Streets would be assigned the name ALMA.

A typical node name would be SNALEXN02. The node is a surface node located on Alexander Street to the north of the lagoons and is the second node defined for the street.

For underground nodes representing the upstream end of a pipe connecting to the trunk drainage system for a street, the suffix A was applied to the node name.

The naming convention adopted in general for links was as follows:

Surface Link name: slaaaaa%%%
Underground Link name: ulaaaaa%%%

Where sl, ul = Surface link and underground link respectively
aaaaa = Abbreviation of location of node.
%%% = Drainage Line number (e.g. 01, 02 etc for each catchment)

Links were generally named in relation to the node at the upstream end of the link. However, for simplicity links were also named in the direction of increasing node number. Although a two digit number was generally used to define the number of the link, in cases where a number of links were related to a single node, a third digit was used in order to ensure that the link could be correlated with its associated node (eg links slalexn01 and slalexn011 are connected to node SNALEXN01).

To avoid confusion with nodes, links were named using lower case letters.

As an example of the link naming convention, link slalexn02 would be a surface link located on Alexander Street to the north of the lagoons and would connect to node SNALEXN02.

The naming convention adopted for links representing Gully pits was as follows:

Gully pit link name: XAAAA%%

Where X = Gully Pit link
AAAA = Abbreviation of location of node
%% = Drainage Line number

In all cases, links representing gully pits were named after the nodes at the surface and the underground system for the connection concerned. For example, link XANNN01 would contain the gully pit details for the connection between surface node SNANNN01 and underground node UNANNN01. Gully pit links were named using upper case letters.

The use of the above naming format allowed the creation of a unique set of node and link names for the model.

The adopted model layout is shown in the following figures in **Appendix B- Model Data**:

Base Cases

• Existing Case	**Figure B1-**	**Network Layout**
	Figure B1A-	**Localised Network Details**
	Figure B1B-	**Localised Network Details**
	Figure B1CONT-	**Contour overlay**
• Ultimate Case	**Figure B2-**	**Network Layout**
	Figure B2A-	**Localised Network Details**
	Figure B2B-	**Localised Network Details**

Relief Drainage Works

• Bideford St (refer Section 4.3)	**Figure B3**
• Frank St (refer Section 4.3)	**Figure B4**
• Macks Road (refer Section 4.4)	**Figure B5**
• Robert Street Option A (refer Section 4.4)	**Figure B6**
• Robert Street Option B (refer Section 4.4)	**Figure B7**
• Robert Street Option C (refer Section 4.4)	**Figure B8**
• Ann Street Option A (refer Section 4.4)	**Figure B9**
• Ann Street Option B (refer Section 4.4)	**Figure B10**

Combined Relief Drainage Works (refer Section 4.7)

• Macks Road to Alexander Street	**Figure B11**
• Frank Street and Bideford Street	**Figure B12**

All of the data necessary to define the links and nodes for the model was created in spreadsheets to facilitate data entry and modification. Each spreadsheet was output as a text file in XPX format (XP Software 1998, pp 110 to 115) and subsequently imported to the XP-UDD model. Separate spreadsheets were created for the surface network, the underground drainage network, and the cross links (i.e. gully inlets) between the surface and underground networks.

3.2 Ultimate Catchment Development

As noted in Section 3.1, consideration was given to existing and ultimate catchment development. For the ultimate catchment model, the existing catchment model was modified as follows:

- *Ultimate catchment development (refer Section 2)*

- *Margaret Street side drainage*

  The drainage works proposed for the streets off Margaret Street (draining to the Margaret Street trunk drainage system) were included in the ultimate case analysis. Relevant details were extracted from Hervey Bay City Council drawing series 2001-082.

- *Alexander Street Lagoon*

  At the time of modelling of ultimate catchment development, the lagoon to the east of Alexander Street was under development. The development requirement to provide 26,500 m^3 of storage between RL 1.5 m and 2.9 m AHD was included in the ultimate case analysis.

  The actual volume of storage available in the lagoon following the completion of development was included in the model of the combined relief drainage works (refer Section 4.7).

- *Additional 1,050 diameter pipe at Elizabeth Street*

  For the ultimate case analysis, it was assumed that the 1,050 stub pipe beneath Elizabeth Street to the south of Dayman Street was extended to provide a total of 3/1,050 diameter pipes draining the catchment to the east of Elizabeth Street.

- *Upgrading Moonbi Street to Parkway Drive system*

  The relief drainage works detailed on Hervey Bay City Council drawing series 2001-36 were included in the ultimate case model.

- *Upgrading Truro Street between Ann Street and Alexander Street*

  The relief drainage works detailed on Hervey Bay City Council drawing series 1025-10 were included in the ultimate case model.

- *Upgrading Truro Street between Margaret Street and Williams Street*

  The relief drainage works detailed on Hervey Bay City Council drawing series 1025-11 were included in the ultimate case model.

- *Upgrading Denmans Camp Road drainage*

  The relief drainage works detailed on Hervey Bay City Council drawing series 2001-153 were included in the ultimate case model.

- *Upgrading Fraser Street outlet and associated works*

  The relief drainage works detailed on Hervey Bay City Council drawing series 451-43 were included in the ultimate case model.

- *Drainage works associated with the link mobility corridor near the point at which Margaret Street and Boat Harbour Drive would join if Margaret Street were to be continued.*

  The drainage works detailed in JWP drawings 1-114098-07 and 08 and 1-114098-42 and 43 were included in the ultimate case model.

- *Addition of Gross Pollutant Traps (GPT's) on all ocean outfalls with diameters equal to or greater than 600 mm.*

  GPT's were added to the Churchill St, Margaret St, Crown St, Ann St, Bideford St, Tavistock St, and Frank St outfalls.

- *Removing the bricks to provide a weir invert level at the inlet to the Churchill St system of RL 1.5 m AHD.*

  At present, a row of bricks is used to maintain an artificially high standing water level in Kondari Resort Lagoon (Lagoon 21, as shown on **Figure 10- Lagoon Improvement Overview**). For the ultimate case analysis, it was assumed that the bricks were removed and the crest level of the weir at the inlet to the Churchill Street system restored to the design level of RL 1.5 m AHD.

3.3 Surface Network

3.3.1 Network Formulation

To correctly account for the available storage within road reserves and drainage channels within the catchment, the catchment was represented by a total of 404 channel links. The location of the links and associated nodes was based on crest and sag locations within the road network and key points within Tooan Tooan Creek and the Lowlands lagoons.

For each link, cross-sections were extracted at a point midway between the upstream and downstream nodes of the link. In all cases, the orientation of sections was assumed to be left to right looking downstream.

For each section, it was possible to define three portions and to assign different Mannings n values to each portion. For the analysis, each section was divided into the left verge, the road (including kerb and channel), and the right verge. For existing roads a Mannings n roughness value of 0.018 was adopted. Verges were considered to have an n value of 0.04.

The general data relating to the surface link network is listed in Table B2- Surface Cross Section Data in **Appendix B- Hydraulic Data**. Adopted surface node invert levels are shown in **Table C1- Peak Water Levels, 2 and 100 Year Events, Surface System** in **Appendix C- Model Results**.

3.3.2 Source of Ground Level Information

The cross sectional information required for the model was obtained from the following sources:

- *Spot levels and contours of the catchment derived from the asset data capture program undertaken by Hervey Bay City Council.*

  The accuracy of the level information collected by the asset data capture program is understood to be 70 mm and therefore quite reliable for flood modelling purposes (Cardno & Davies 1998). This level information was correlated with the information collected regarding pipe inverts to provide details of surface levels within the study area.

- *Survey completed for Barlow Gregg Abercromby & Associates' Drainage Strategy for Drainage of Catchment Entering Tooan Tooan Creek (January 1990).*

  For the study, a number of road crossings were surveyed together with some drain cross sections.

- *Survey commissioned by Barlow Gregg and Associates for their study of the Pialba catchment*

  Survey for the region between Zephyr Street and Charlton Esplanade was surveyed for the study being undertaken by BGA.

- *Survey commissioned for this investigation*

  At the commencement of the study, the available survey data was reviewed and any gaps in the data identified. In particular, information in relation to road and driveway crossings of Tooan Tooan Creek and the lagoons themselves was found to be required. A brief for additional survey was subsequently prepared and detailed survey undertaken by Wide Bay Engineering Services. A total of 52 cross sections were surveyed for the study.

3.3.3 Weir Relations

During significant flood events, it is likely that road crossings would be overtopped. To account for this and overland flow resulting from ponding in sags, a total of 62 weirs were added to the model. Two options were available with respect to the definition of the weirs:

- Standard weir, where a crest width and elevation are specified, and
- Special Weir, where the flow characteristics of the weir are specified for a range of flow depths.

Although the standard weir approach is more straightforward, only a single weir crest level can be input. In cases involving roads, the crest width widens as the depth of flow over the weir increases. For this reason, it was considered preferable to use the more complex Special Weir formulation due to the freedom it provided in relation to the specification of weir flow characteristics for a range of depths.

For each weir, the width of flow at a number of levels was calculated and input to the XP-UDD model. The only complication associated with the use of the Special Weir formulation is that levels are specified as depths above the invert level of the upstream node to which the weir is attached.

Consequently, the levels at which weir widths were calculated had to be converted to equivalent depths. To prevent flow occurring before the actual crest level of the road was reached, zero length values were entered for zero depth (i.e. the invert level of the upstream node) and the depth at which weir flow commenced. The correct weir length was then entered for a point 0.001 m above the depth at which flow commenced. The depth level relationship defined for each weir is presented in **Table 3.2-Weir Relations-Initial Values**.

In order to minimise instabilities at the point at which overflow first commenced, the coefficient of flow was reduced from 1.7 to 1.3 for the first 50 mm of flow depth.

TABLE 3.2
Weir Relations- Initial Values

Level (m AHD)	Depth above Upstream Node Invert (m)	Weir Length	Coefficient of Flow
Node Invert level	0	0	0.1
Level at which flow commences	Depth at which flow commences	0	0.1
Level at which flow commences + 0.001	Depth at which flow commences + 0.001	Length for level	1.3
Level at which flow commences + 0.050	Depth at which flow commences + 0.050	Length for level	1.7

Details of the surface weir links are listed in **Appendix B- Hydraulic Data**.

3.3.4 Lagoon Areas

To ensure that the storage afforded by the Lowlands Lagoons was correctly accounted for in the model, storage areas were added to nodes in the model as required. Care was taken to ensure that the overall volume of the lagoons was not over-estimated. For each lagoon, the volume already represented in the model via links within the lagoon was deducted from the overall lagoon volume and the remaining volume added to the model.

The lagoon areas adopted for use in the study are listed in **Table 3.3- Lagoon Volumes**.

The lagoon areas were adjusted as part of the relief drainage works to account for future enlargement works (refer Section 4.2). The areas adopted for this case are listed in the column 'Maximised Volumes' in Table 3.3.

The extent of enlargement possible within the lagoons was revisited following the initial analysis of relief drainage options. Due to the encroachment of development, it was recognised that it may not be possible to achieve the enlargement initially envisaged. The flood storage volume adopted for each of the lagoons was therefore revised to reflect the works that could be readily achieved. These volumes, shown in the column 'Achievable Volumes' in Table 3.3, were used in the analysis of the impact of the combined relief drainage works (refer Section 4.7).

It can be noted that the volumes presented for Lagoon 20 (Botanic Gardens) include the entire storage volume available within the gardens between Margaret Street and Kondari Resort.

TABLE 3.3
Lagoon Volumes

Lagoon	Ultimate BaseCase (m^3)	Maximised Volumes (m^3)	Achievable Volumes (m^3)
Lagoon 73- Northern Lagoon west of Margaret Street	22,200	22,810	22,810
Lagoon 60- Northern Lagoon between Ann Street and Alexander Street	46,300	50,700	48,600
Lagoon 50- Lagoon between Robert Street and Ann Street	51,300	61,300	54,500
Lagoon 40- Southern Lagoon between Ann Street and Alexander Street	18,100	24,900	22,000
Lagoon 30- Southern Lagoon between Alexander Street and Margaret Street	35,500	34,200	34,200
Lagoon 20- Botanic Gardens including Richard Street	137,500	143,000	143,000
Lagoon 21- Kondari Resort	43,400	41,200	41,200
Lagoon to East of Alexander Street	25,900	26,500	21,200
Lagoon to West of Robert Street	-	-	8,100

Note: Refer to Figure 10- Lagoon Improvement Overview for location of lagoons

3.3.5 Inlets to Margaret and Churchill Street Drainage Systems

Major drainage systems constructed in Margaret and Churchill Streets provide for the drainage of the Lowlands Lagoons to Hervey Bay. The crest level of the weir inlets to both systems determines the standing water level in the lagoon system.

Details of the inlet weirs were obtained from Drawings 2001-082-5 (Margaret Street) and 2001-049-5 (Churchill Street). The crest level shown on the drawings in both cases is RL 1.5 m AHD.

3.4 Underground Network

3.4.1 General

The underground stormwater drainage system was modelled using closed conduit links. Council, as part of its Asset Data Capture program, collected information relating to the existing drainage network. This information was used to recreate the existing drainage system in the XP-UDD model.

In addition to the inclusion of links to represent the main drainage lines, links were also added at points of major inflow to the stormwater system to allow the effect of pipes connecting gully pits to the main drainage line to be accounted for when considering the acceptability of calculated water levels.

A total of 245 links were added to the model to account for the underground drainage system. A Mannings "n" value of 0.013 was adopted for all pipes. All pertinent data relating to the underground drainage network for the existing and ultimate development cases are presented in **Appendix B- Hydraulic Data** in **Table B1- Conduit Data Existing Development** and **Table B5- Conduit Data Ultimate Development**.

The tables include information on the length, diameter and number of pipes, invert levels, node connectivity information and adopted junction loss values.

3.4.2 Manhole Losses

Although loss coefficients can be input to account for manhole losses, the Hydraulic (EXTRAN) mode of XP-UDD is not capable of dynamically altering manhole losses to suit flow conditions. Only a single coefficient (multiplied by the velocity head in the downstream pipe) can be input for each pipe to represent losses at junctions throughout a storm event.

In order to assign appropriate manhole loss coefficients, recourse was made to the generalised values presented by Argue in the publication *Storm Drainage Design in Small Urban Catchments- A Handbook for Australian Practice* (1986). Argue recommends single values for given junction scenarios, eliminating the need for extensive reference to charts as required under the *Queensland Urban Drainage Manual* (QUDM)(Neville Jones & Associates et al 1992, Section 5 and Volume 2). When comparing designs completed using his simplified values and designs completed using the Missouri Charts (Sangster et al 1958, as quoted in Argue 1986, p 45), Argue found that the resultant difference between the designs was minimal (Argue 1986, p 106).

The values proposed by Argue (Argue 1986, pp 46-48) were compared with values likely to be derived from the QUDM charts and a generally conservative amalgam of junction pit loss coefficient values derived. The manhole loss coefficients adopted for the investigation for various pipe configuration scenarios are listed in **Appendix B- Hydraulic Data** in **Table B8- Adopted Manhole Loss Coefficients**.

3.5 Stormwater Inlet Pits

3.5.1 General

To model stormwater inlet pits and the interaction between the surface and underground drainage networks during surcharge conditions in the pipe system, stormwater inlet pits were modelled as weirs via the multiple conduit/ diversion option in the XP-UDD model (XP Software 1998, pp 243-244). Inlet pits were modelled using weirs rather than rating curves due to the ability of the Hydraulic (EXTRAN) mode of XP-UDD to model the drowning of weirs when the downstream water level approaches the upstream water level, thereby allowing the simulation of surcharge conditions and restricted inflow conditions during major storm events.

The required weir relationships were derived by first considering the inlet capacity of each of the various types of inlet pit present within the catchment. Hervey Bay City Council standard 1 bay, 2 bay, 3 bay and 4 bay side entry pits (Hervey Bay City Council standard drawings SD 010 and SD 011 Revision 0) were considered together with grated inlets. It can be noted that side entry pits were included for all new drainage lines analysed. For all types of inlet, separate inlet capacity curves were derived for sag and on grade conditions.

The inlet capacity curves were then converted to equivalent weir relationships by calculating the length of weir required at each defined water level to produce a flow equivalent to the inlet capacity for that level.

For weir relations, XP-UDD requires that the weir length, coefficient and discharge exponent be specified for each depth entered in the relation.

It was found that the standard value of discharge exponent (1.5) produced a significant variation in the weir length required with increasing depth. To minimise the variation in weir length with depth (and to thereby reduce the potential for instabilities in the model), a discharge exponent of 0.2 was adopted. A standard weir discharge coefficient of 1.7 was adopted for all water depths.

Further, the weir tables had to be adjusted at very small depths because the low exponent value generated appreciable flows at very small depths over the weir. This was overcome by using weir parameters for flow depths to 0.001 m (1 mm head on the weir) as listed in **Table 7- Inflow Rating Curves- Weir Correction Parameters**.

TABLE 3.4
Inflow Rating Curves - Weir Correction Parameters

Head on weir (m)	Weir length (m)	Exponent	Coefficient
0.0	As for head 0.08	1.5	0.1
0.001	As for head 0.08	1.0	1.0
0.08	Calculated	0.2	1.7

The adopted inlet capacity curves are presented in **Appendix B- Hydraulic Data** in **Table B9- Inlet Capacity of Side Entry Pits, Grates and Field Inlets** and **Table B10- Equivalent Weir Dimensions for Inlet Pits**.

Due to the number of inlet pits present within the catchment, inlet pits draining to the same node on the underground drainage system were aggregated and modelled as a single inlet.

The weir relation used to define flow into the underground drainage system was also used to define flow to the surface in the event of surcharge conditions within the pipe network. Due to current limitations of the Hydraulic (EXTRAN) mode of XP-UDD, it is not possible to model one directional weirs. Such a facility would have allowed the specification of separate relations for flow into and out of the underground system. Other methods to provide the desired replication of flow conditions were not able to be justified due to the prohibitively large number of links and nodes that would be required. A review of the surcharge capacity of inlets indicated that the use of inflow capacity curves underestimated the outflow capacity to some degree. This was considered to be acceptable given the need to limit the number of links used in the model.

3.5.2 Sag Inlets

The Hervey Bay City Council side entry pit capacity curves for sag conditions are presently based upon the United States Department of Transportation formula for weir flow. At higher depths above the invert level of the pit, the capacity of inlets will be limited by orifice type flow (Neville Jones & Associates et al 1992, p 5-47). Based on the available orifice area for each of the various side entry pit types (calculated from Hervey Bay City Council standard drawings), curves were produced for each type of side entry pit for weir and orifice type flow conditions and the resulting flow envelope adopted.

It can be noted that the inlet curves for side entry pits are not based on model tests of the actual Council design. Full size model testing of Brisbane City Council gully inlets by the Urban Water Resources Centre, University of South Australia (Argue 1994, as quoted in Brisbane City Council 1994, Chart QUDM-1) has indicated that the capture rate of inlet pits when the water level is about 90 mm above the top of the kerb is significantly less than that predicted by the orifice equation.

Although the orifice equation and normal hydraulic considerations indicate that the flow into an inlet should increase with increasing water level, the results of model testing led to the recommendation that the capacity of inlets not be extrapolated beyond a certain point. A similar result was obtained from testing of the Rocla "Drainway" unit (Argue 1993, as quoted in QUDM Volume 2).

Given the experimental results, the inflow relations for inlet pits were conservatively limited at depth, to reflect the outcome of model testing of other pit types.

Based on the orifice area of the Hervey Bay side entry pits and the orifice area of the Brisbane City Council and Drainway inlets, the following limiting flow capacities were conservatively adopted for the study:

- 1 Bay 270 L/s
- 2 Bay 310 L/s
- 3 Bay 375 L/s
- 4 Bay 475 L/s

The calculated sag capacities were reduced by 20 percent to account for blockage effects (Hervey Bay City Council 1997, p 16-9, Neville Jones & Associates 1992, p 5-42).

For grated inlets to the existing drainage system, a grate with a gross area of 0.5 m^2 was assumed. This area was reduced by 25 percent to account for the reduction in waterway area caused by the bars of the grate. The inlet capacity of the grate was defined according to the weir and orifice equations (Neville Jones & Associates 1992, pp 5-47 to 5-48). The calculated capacity of the grate inlet was reduced by 50 percent to account for blockage effects (Hervey Bay City Council 1997, p 16-9, Neville Jones & Associates 1992, p 5-42).

3.5.3 On Grade Inlets

The inlet capacities of on grade side entry pits and grates were derived from the Hervey Bay City Council gully pit capacity charts which relate captured flow to gutter flow. In order to provide the required level-inflow capacity relation, the water depth associated with various gutter flows was determined.

The water level for a given gutter flow varies according to the longitudinal and cross slope of the road and road width. To provide typical water levels for the types of road present within the study area, roads with widths of 8 metres and 12 metres and longitudinal slopes of between 0.25 percent and five percent were analysed using the backwater program HEC-RAS. The roads were assumed to have a cross fall of 1 in 30 and to lie within a 20 metre road reserve. The results of the analysis were used to assign water levels to a range of gutter flows and thereby establish the necessary stage discharge relations for on grade side entry pits and grates.

The calculated on grade capacities were reduced by 20 percent in the case of side entry pits and 50 percent in the case of grates to account for blockage effects (Hervey Bay City Council 1997, p 16-9, Neville Jones & Associates 1992, p 5-42).

3.5.4 Field Inlets

The Lowlands Lagoon catchment features a number of field inlets ranging in size from 230 mm diameter to 600 mm diameter.

Suitable inflow relations for the inlets were obtained using the generalised relationships proposed by Boyd (1987, p161-165).

For the analysis of the relief drainage works constructed in Robert Street (refer Section 4.7), suitable relations for the Webforge field inlets specified on the design drawings for the works.

3.6 Tailwater Levels And Initial Model Conditions

The following tailwater levels were adopted for the analysis (Hervey Bay City Council 1997, p 16-14):

- Minor Storm Event (2 and 10 Year) RL 1.50 m AHD (MHWS + 0.3 m Greenhouse Effect).

- Major Storm Event (20, 50, and 100 Year) RL 2.40 m AHD (HAT + 0.3 m Greenhouse Effect).

Normal practice when completing dynamic models is to run the model for a number of hours with small inflows to enable the model to calculate initial water levels and flows for use when modelling design events. This leads to the creation of hotstart files for use in design runs. For the run, tailwater levels are typically set at the level to be adopted for the design event. The hotstart approach removes the majority of instabilities at the start of design event runs.

In the case of the Lowlands Lagoons, special care was paid to the initial conditions in the system at the commencement of design events. As noted above, for the major storm event, the tailwater level is RL 2.4 m AHD. An initial run with this level would have led to water moving upstream from Hervey Bay and filling the lagoons to a level of RL 2.4 m AHD. As the storage available in the lagoons is a major determinant of peak flood levels, the level assumed in the lagoons has to match the nominal level in the lagoons at the start of a design event.

Further, although it is conceivable that flow could enter the lagoons from the bay via those stormwater systems without flapgates, it is unreasonable to expect that the tailwater level would be maintained at a constant level for a sufficient period for this to occur in reality.

Therefore, to obtain a set of reasonable levels in the lagoons at the start of design events, the following methodology was applied:

- Complete a short run with tailwater levels set at RL 2.4 m AHD. This provided a level of RL 2.4 m AHD in the lagoons.
- Complete a second run with a tailwater level set at RL 1.5 m AHD using the conditions calculated at the end of the first run. The lagoons then drained slowly to their nominal standing water level.

The hotstart file created at the end of the second run was used to provide the initial conditions for both minor and major design event runs.

3.7 Model Parameters

For the analysis, a relatively short time step of one second was adopted due to the complexity of the model and in order to promote stability. Model run times for major event storms were found to be considerably longer than those required for minor storm events. This was attributed to the drowning of inlet pits during major flow conditions and the model having to complete a significant number of iterations to produce a solution at each time step in these cases.

A tolerance of 0.001 m^3/s and 0.001 m was used with respect to the convergence of flows and levels respectively at each time step. The maximum number of iterations for any one time step was set at 500.

The period of time modelled for each storm duration is listed in Table 3.5.

TABLE 3.5
Period of Time Modelled

Storm Duration	2 Year Events	100 Year Events
15 minutes	7 hours	7 hours
20 minutes	7 hours	7 hours
25 minutes	7 hours	7 hours
30 minutes	7 hours	7 hours
45 minutes	7 hours	7 hours
1 hour	7 hours	7 hours
90 minutes	7.5 hours	7.5 hours
2 hour	8 hours	8 hours
3 hour	9 hours	9 hours
4.5 hour	10 hours	10 hours
6 hour	12 hours	12 hours
9 hour	14 hours	14 hours
12 hour	17 hours	17 hours
18 hour	21 hours	24 hours
24 hour	28 hours	30 hours
30 hour	33 hours	37 hours
36 hour	39 hours	43 hours

3.8 Sensitivity Analyses

3.8.1 General

To test the validity of the model, a number of sensitivity analyses were undertaken. The following sections describe the analyses undertaken and their potential overall impact on predicted levels.

3.8.2 Groundwater Impacts

For the analysis to date, no account was taken of the underlying sand aquifer storage and transfer capacity. The main reason for this was to reduce the overall cost of the study

One of the key pieces of information required in order to determine the amount of groundwater inflow is the storm duration that produces the peak level in the lagoon system. For short duration storms, the amount of infiltration and underground flow that could occur is relatively limited. However, as the storm duration increases, the potential for infiltration also increases due to the increased length of time water is stored in each of the lagoons.

The flood study determined that the 30 hour storm produced peak levels within the lagoons. Based on an assumed groundwater level of RL 1.5 m AHD (equivalent to the nominal level in the lagoons) and using the time to peak predicted by XP-UDD together with likely soil parameters, the potential loss from the lagoons due to groundwater infiltration was estimated. The calculated volume was then converted to an equivalent change in the peak level in the lagoon.

Although the amount of infiltration that will occur varies according to soil conditions, it is estimated that for the 100 year event the maximum level in the lagoon would be reduced by between 1 mm and 30 mm, with a likely reduction of the order of 12 mm. Although not considering the effects of infiltration will lead to a conservative estimate of flood levels, it is considered that the likely magnitude of groundwater infiltration will be minimal in comparison to the peak level reached in the lagoons.

In addition to infiltration in the vicinity of the lagoons, there is also the potential for infiltration to occur within individual allotments, thereby reducing the amount of runoff from properties. However, given the high percentage of area within developed blocks that is both impervious and directly connected to the stormwater drainage system, the potential for infiltration to occur is limited. The potential is further limited by the introduction of topsoil on blocks. Given these factors, it was considered that the existing infiltration potential within developed blocks is relatively limited. In any case, the use of runoff coefficients in the derivation of peak flows allows for some infiltration loss for minor events. Although the loss is minimal for the 100 year event, it was considered that in such situations that the catchment could be approaching saturation so the potential for infiltration is minimal.

Consequently, it was concluded that the modelling approach adopted for the study was reasonable and not overly conservative.

3.8.3 Maintenance of Standing Water Level in Lagoon and Storm Surge

A key assumption in the design of the lagoons is that a standing water level of RL 1.5 m AHD can be maintained in the lagoons. Floodgates have been provided on the Margaret and Churchill St outlets to ensure that high tides do not cause the level in the lagoons to rise. For major event storms, a tailwater level of RL 2.4 m AHD is typically adopted. Provided that the level in the lagoons is close to RL 1.5 m AHD at the start of the event, then a considerable storage volume will be available between RL 1.5 m AHD and RL 2.9 m AHD. The storage volume will provide a reduction in peak lagoon level and therefore levels within the catchment.

Although flood gates are provided at Margaret and Churchill Streets, there are no flood gates on the main creek itself. A storm surge could propagate up the creek and fill the lagoons prior to the commencement of runoff. As the duration of the storm surge and the length of creek between the mouth of Tooan Tooan Creek and the lagoons is relatively long, the level in the lagoons will not increase to match the peak storm surge level.

Analysis of Storm Surge Penetration

To determine the impact of storm surge on lagoon levels, the calculated storm surge profile for the 100 year event derived by the recent Lawson and Treloar storm surge study was applied as a boundary condition to the model and the model run without any runoff from the catchment.

The modelling provided the following peak levels in the lagoons:

- Central RL 1.83 m AHD (i.e. 330 mm above standing water level)
- Northern RL 1.63 m AHD
- Southern RL 1.51 m AHD

Based on this, storm surge could be expected to reduce the available storage within the lagoons by less than 20 percent. The attenuation afforded by the main creek, particularly in the region near Robert Street where ground levels are relatively high, is sufficient to minimise intrusion by storm surge.

Storm Surge Coinciding with Peak Runoff

Given the proximity of the catchment to the ocean, there is some likelihood that storm surge would coincide with the peak rainfall across the catchment. To quantify the potential impact of storm surge combined with rainfall, the surge only analysis was followed by the consideration of the combined effect of storm surge and a 100 year 30 hour storm (the duration which produces the peak level in the lagoon system).

In the main channel to the west of the lagoons, peak levels increased due to the increase in peak level adopted at the outlet of Tooan Tooan Creek. It can be noted that the peak storm surge level (including greenhouse) is RL 3.2 m AHD, compared to the level of RL 2.4 m AHD normally adopted for major events. Within the lagoons themselves, the following changes in peak level were obtained:

- Central No change in level
- Northern 18 mm increase in level
- Southern 30 mm reduction in level

The relatively small increase and the decrease in level can be attributed to the variation in tailwater level included in the storm surge analysis and the long (30 hour) critical storm duration in the lagoons. Although the storm surge produces a peak level which is higher than the major event tailwater level, the duration of the peak is relatively short, with water able to discharge from the lagoons relatively easily during the low tide conditions on either side of the peak. The relatively conservative assumption of a constant tailwater level for major events, particularly for longer duration storms, results in the calculation of peak levels in the lagoons that are similar to those obtained using a more realistic hydrograph.

Given this result, it was concluded that the results obtained using the constant tailwater level for major events were reasonable, and unlikely to be altered by more than about 20 mm by storm surge effects.

Antecedent Rainfall

The water level in the lagoons may also be affected by rainfall which occurs in the period prior to the commencement of the design event. Although such rainfall is not typically modelled when considering design events, during real floods there will usually be a period of hours or days of rainfall preceding the main rainfall burst.

For modelling, a constant tailwater level of RL 2.4 m AHD was assumed. Based on this assumption, any runoff entering the lagoons (standing water level RL 1.5 m AHD) prior to the main burst could not be discharged and would therefore reduce the storage available during the main burst. In reality, water collected in the lagoons could be discharged during low tides. Consequently, only runoff occurring in the period immediately preceding the main rainfall burst (and during high tide conditions) could potentially be locked in the lagoons and reduce the available storage.

It is estimated that about 55 mm of runoff would be required to fill the lagoons to a level of RL 2.4 m AHD. It is difficult to envisage such an amount of rainfall occurring within a sufficiently short period and in close enough proximity in time to the main burst without being a minor storm event in its own right. In such cases, it could be argued that the combined probability of a minor event and a major event occurring in such close proximity is greater than that of the major event by itself.

To determine the potential impact of antecedent rainfall on flood levels, as a worst case scenario it was assumed that the lagoons were full to RL 2.4 m AHD at the start of the 100 year 30 hour storm event. Calculated peak levels in the lagoons increased by the following amounts:

- Central 61 mm
- Northern 116 mm
- Southern 65 mm

The reduction in available storage also increased the peak level in the main creek west of the lagoons by between 50 and 60 mm. Further, levels in the Margaret Street drainage path were also found to increase by 116 mm or more.

It must be stressed that the above is a worst case scenario, assuming that the entire storage volume of the lagoons is full to RL 2.4 m AHD at the commencement of the flood event. For the reasons nominated above it is unlikely that the lagoons would be filled to such a level. Overall, it is considered that antecendent rainfall could produce levels no more than 30 to 50 mm greater than those calculated assuming a constant tailwater and no antecedent rainfall.

Summary

Analysis has indicated that it is reasonable to adopt a standing water level in the lagoons of RL 1.5 m AHD at the start of major rainfall events. Although storm surge penetration and antecedent rainfall could increase the water level in the lagoons prior to the commencement of the main rainfall event, it is estimated that the impact of the reduction in available storage would affect peak levels by at most 30 to 50 mm.

Such a margin on peak flood levels is considered to be minimal.

3.9 Existing and Ultimate Base Case Results

Detailed results for the existing and ultimate catchment base cases are presented in the following tables in **Appendix C- Model Results**:

Table C1	**Peak Water Levels, 2 and 100 Year Events, Surface System, Existing Development, Ultimate Development and Adopted Mitigation Works Scenarios**
Table C2	**Peak Water Levels, 2 and 100 Year Events, Underground System, Existing Development, Ultimate Development and Adopted Mitigation Works Scenarios**
Table C3	**Peak Flows and Velocities, 2 and 100 Year Events, Surface System, Existing Development, Ultimate Development and Adopted Mitigation Works Scenarios**
Table C4	**Peak Flows and Velocities, 2 and 100 Year Events, Underground System, Existing Development, Ultimate Development and Adopted Mitigation Works Scenarios**

The extent of inundation calculated for the ultimate base case is presented in **Figure 2- 2 Year Flood Map** and **Figure 3- 100 Year Flood Map**.

3.10 Modelling of Relief Drainage Works

The results obtained for the ultimate base case were reviewed and areas subject to flooding identified. A range of relief drainage works, as described in Section 4, were then considered.

For each case considered, the ultimate base case model was adjusted to reflect the proposed works. The changes made to the model to represent each of the relief drainage scenarios are listed in **Appendix B- Hydraulic Data**.

4.0 RELIEF DRAINAGE OPTIONS

4.1 Design Criteria

The results obtained from the existing and ultimate base case analyses were reviewed to gain an understanding of the flow distribution and flooding problem areas within the Lowlands catchment. Available options for flood mitigation were then considered and modelled.

For relief drainage works of the type considered in the investigation, the *Queensland Urban Drainage Manual* (QUDM) (Neville Jones & Associates et al 1992, p11-3) notes that:

"Whilst the criteria set down in the Manual should be adhered to if possible for relief drainage works, economic and physical limitations may require the adoption of less stringent criteria."

Consequently, the philosophy adopted for the sizing of relief drainage works was to obtain a solution which complied with QUDM except in cases where the cost of the works would be prohibitively high compared to the benefit obtained or where physical constraints (eg insufficient cover) precluded the adoption of a cost effective solution.

The following design criteria were adopted for the investigation:

DRAINAGE DESIGN CRITERIA		
Minor (2 Year) Events		
Pipe Drainage:	In General:	Freeboard to surface invert > 150 mm (QUDM standard). No surcharge within existing drainage lines connected to new drainage lines.
	Worst Case:	Water level within 150 mm of surface invert. No surcharging allowed on new drainage lines (relief trunk drainage works excepted).
Surface Drainage:	In General:	Water levels in road system and above gully pits to be not greater than the top of kerb level.
	Worst Case:	Maximum depth of water not to exceed 250mm in areas serviced by new drainage lines.
Major (100 Year) Events		
Surface Drainage:	In General:	Water depth not to exceed 270 mm in road reserves (QUDM standard), with target depth of 300 mm adopted in cases where QUDM standard cannot be achieved.
		Velocity Depth product not to exceed 0.6 m^2/s
		Target Lagoon and Tooan Tooan Creek peak flood level RL 2.9 m AHD.
	Worst Case:	Case by Case basis with maximum depth of 600 mm in road reserves.
		Velocity Depth product not to exceed 0.9 m^2/s.

When considering available relief drainage options, it was recognised that adding new beach outfalls would not be acceptable. Where practicable, it would be desirable to remove existing outfalls.

With the exception of the Margaret and Churchill Street outfalls, the fifteen other existing outfalls in the Lowlands catchment do not extend across the beach. The relatively high invert levels of the existing pipes allow them to be terminated near the frontal dune. Flow discharges from the pipes across the beach. Although it was considered that these existing outfalls could be upgraded, significantly increasing the extent to which the outfalls extended across the beach would not be acceptable. Consequently, invert levels close to those of the existing pipes were adopted when considering upgrading options. In a number of cases this provided a constraint to the size of pipe that could be accommodated upstream given minimum cover requirements.

For the investigation, it was considered acceptable for the existing Margaret Street and Churchill Street outfalls to be augmented. These outfalls discharge to the ocean via rock groynes. Increasing the width of the groynes by adding pipes to the outfalls would have an insignificant impact on their present visual amenity and the present impact of the groynes on coastal processes.

All mitigation options were based on the ultimate development base case model of the catchment. The 2 year minor storm event and 100 year major storm event were considered for the analysis. Storm durations ranging from 15 minutes to 36 hours were considered. The cost of each mitigation measure (excluding GST) is noted with the description of the impact of the works. Detailed costing information is presented in **Appendix D- Costing of Relief Works**.

Sections 4.2 to 4.6 detail the relief drainage measures considered within the catchment together with the constraints and benefits associated with each measure. The location of each of the relief drainage works is shown on **Figure 4- Relief Drainage Works Key Plan.**

Following a review of the available relief drainage options, Council indicated those works most likely to be constructed. The effect of these works in combination was modelled to determine the overall benefit to be obtained from the completion of relief drainage works within the catchment. The measures modelled and the results of this analysis are presented in Section 4.7. It can be noted that the various works modelled for the combination case were, in a number of cases, modified on the advice of Council to reflect the works that could actually be accomplished in practice.

4.2 Lagoon Drainage Options

The Lowlands Lagoon system comprises a total of seven large lagoon areas divided by road crossings. The lagoons can be grouped as follows (refer **Figure 10- Lagoon Improvement Overview**):

- *Northern Lagoons*
  - Lagoon 73 and the lagoon area bounded by Margaret Street and Alexander Street
  - Lagoon 60, bounded by Alexander Street and Ann Street

- *Central Lagoon*
  - Lagoon 50, bounded by Ann Street and Robert Street

- *Southern and Eastern Lagoons*
  - Lagoon 40, bounded by Ann Street and Alexander Street
  - Lagoon 30, bounded by Alexander Street and Margaret Street
  - Lagoon 20, located in the Botanic Gardens and bounded by Margaret Street and Kondari Resort
  - Lagoon 21, Kondari Resort

During large storm events, the three main mechanisms by which the lagoons are drained are as follows:

- *The trunk drainage system in Margaret Street that drains water in Lagoon 73 to the ocean.*

  A weir at the upstream end of the system provides a nominal standing water level in the lagoon system of RL 1.5 m AHD. Flapgates have been fitted to the Margaret Street system to prevent the flow of water from the ocean to the lagoon.

  The invert level of the Margaret Street pipes at their outlet is RL –0.6 m AHD. To achieve this level, it was necessary to construct a rock groyne extending over 50 metres beyond the Mean High Water Spring tide level.

- *The trunk drainage system in Churchill Street which drains water in the Kondari Resort lagoon to the ocean.*

  Similar to Margaret Street, an inlet weir at the upstream end of the system provides a nominal standing water level in the lagoon of RL 1.5 m AHD and floodgates prevent the flow of water from the ocean to the lagoons. A rock groyne similar to that constructed at Margaret Street was necessary to achieve the required invert level at the outlet of the pipes.

  The efficiency of the Churchill Street system is limited by the relatively high ground levels in the Botanic Gardens (i.e. surrounding Lagoon 20) that preclude the drainage of the lagoons to the west via Churchill Street during minor events.

- *Tooan Tooan Creek.*

  Immediately to the west of Robert Street, Tooan Tooan Creek is poorly defined. Further downstream, the channel is relatively narrow. These factors limit the capacity for flow to be discharged from the Lagoons.

For the ultimate development scenario, the peak water level in the lagoons was found to vary from RL 2.9 m AHD in the northern lagoons to just over RL 3.0 m AHD in the central, southern and eastern lagoons.

The following relief drainage options for reducing the peak water level reached in the lagoons were considered:

- ***Bund at Robert Street (refer Figure 5- Robert Street Bund and Lake)***

  When reviewing the results obtained for the present situation, it was noted that a considerable volume of water entered the lagoon system from the sub catchments to the west of Robert Street. The majority of this water is drained via the Margaret and Churchill Street outfalls. As the volume of water entering the lagoons influences considerably the peak level reached in the lagoons, reducing the volume of water entering the lagoons would provide an immediate flood level reduction, particularly for minor event flooding.

  As the existing ground levels in the vicinity of Robert Street are relatively high, a bund would be relatively cheap to construct. However, the runoff diverted from the lagoons would need to be drained via relief drainage works in the western part of the catchment in order to avoid increasing flood levels in that portion of the catchment (refer Section 4.3).

- ***Maximise Lagoon Storage Area (refer Figure 10- Lagoon Improvement Overview)***

  The 1997 *Retention Basin Extension Study* (Cardno & Davies) completed for Council considered the potential for maximizing the storage area available within the Lagoons (Hervey Bay City Council drawings 2001-095 C1 to C9). Development around the lagoons subsequent to the completion of the study limits the extent to which extension works can now be completed.

  Further, the creation of deep lagoons within the Botanic Garden area (Lagoon 20) would not be acceptable due to concerns in relation to groundwater and ecological impacts. However, it would be possible to construct a high level channel within the Gardens which would provide improved hydraulic connectivity between the Kondari Resort lagoon and the remainder of the lagoons.

  Providing increased conveyance across the Gardens would allow a greater amount of flow to be discharged via the Churchill Street outfall and maximize the benefit to be obtained from augmenting the outfall.

  Consequently, although the potential reduction in flood level afforded by the enlargement of the lagoons may not be as great as initially envisaged, the link between the lagoons and the Churchill Street outfall provided by the work would afford a considerable reduction in flood level throughout the lagoon system.

- ***Enlarging Lagoon Area by Works West of Robert Street (refer Figure 5- Robert Street Bund and Lake)***

  The potential exists to excavate a lake within the lot to the west of Robert Street to maximize the flood storage capacity of the lagoons. For the analysis, it was assumed that the majority of the lot would be converted to a lagoon with a standing water level of RL 1.5 m AHD, increasing the flood storage volume available within the lot (between RL 1.5 m AHD and RL 2.9 m AHD) by 8,100 m^3 compared to the existing situation.

- ***Augmenting Margaret Street Outfall (refer Figure 6- Margaret Street Augmentation)***

  The existing drainage system at Margaret Street could be increased in size to improve the drainage of the lagoons. However, as the distance between the inlet weir and the outlet of the system at Margaret Street is significantly greater than the corresponding distance at Churchill Street, it was considered more cost effective to increase the size of the Churchill Street drainage system provided a suitable link could be provided across the Botanic Gardens.

  The existing drainage at Margaret Street consists of twin 1,350 mm diameter RCP's between the lagoon and Cypress Street, increasing to three 1,350 mm diameter pipes downstream of Cypress Street. Given the greater cost effectiveness of providing additional drainage at Churchill Street, the modelling of additional drainage at Margaret Street was limited to increasing the number of pipes to three 1,350 mm diameter pipes for the length of the Margaret Street system.

- ***Augmenting Churchill Street Outfall (refer Figure 7- Churchill Street Augmentation)***

  By virtue of the relatively short distance between the Kondari Resort lagoon and the ocean, providing additional drainage at Churchill Street is an attractive option. For the analysis, it was assumed that the existing system (2,400 x 1,400 RCBC) would be duplicated.

In addition to the above options, consideration was given to widening Tooan Tooan Creek to improve its conveyance, particularly in the reach between Robert Street and downstream of Fraser Street. However, the following factors indicated that such works would not be acceptable:

- Flood levels are already high in Tooan Tooan Creek. Directing more flow to the creek, even if it were to be widened, would be unlikely to provide any relief from flooding in the western part of the catchment, and
- The relatively high ground levels in the area downstream (i.e. west) of Robert Street limit the extent of penetration of storm surge from the mouth of Tooan Tooan Creek. The excavation of a wide channel downstream of the lagoons would lead to increased storm surge impacts in inland areas.

The following combinations of the above drainage options were modelled:

1) Bund at Robert Street
2) Enlarged lagoons
3) Bund at Robert Street together with enlarged lagoons
4) Bund at Robert Street together with enlarged lagoons and additional lagoon area to the west of Robert Street (assuming that the bund would be located so as to allow connection of the additional lake area to the Central Lagoon (Lagoon 50).
5) Bund at Robert Street and augmentation of Margaret Street Drainage system
6) Augmentation of Churchill Street
7) Bund at Robert Street and augmentation of Churchill Street
8) Bund at Robert Street together with enlarged lagoons and augmentation of Churchill Street.

The peak flood levels within the lagoons and at key points in Tooan Tooan Creek are summarized in the following tables:

• Options 1) to 4)	**Table 4.1**	**2 Year Event, Relief Drainage Options**
	Table 4.2	**100 Year Event, Relief Drainage Options**
• Options 5) to 8)	**Table 4.3**	**2 Year Event, Upgrading Margaret and Churchill Streets**
	Table 4.4	**100 Year Event, Upgrading Margaret and Churchill Streets**

Detailed results for the analysis are presented in **Appendix C- Model Results**:

- *Options 1) to 4)*

Table C5	**Peak Water Levels, 2 Year Event, Surface System**
Table C6	**Peak Water Levels, 100 Year Event, Surface System**
Table C7	**Peak Water Levels, 2 Year Event, Underground System**
Table C8	**Peak Water Levels, 100 Year Event, Underground System**
Table C9	**Peak Flows and Velocities, 2 Year Event, Surface System**
Table C10	**Peak Flows and Velocities, 100 Year Event, Surface System**
Table C11	**Peak Flows and Velocities, 2 Year Event, Underground System**
Table C12	**Peak Flows and Velocities, 100 Year Event, Underground System**

- *Options 5) to 8)*

Table C13	**Peak Water Levels, 2 Year Event, Surface System**
Table C14	**Peak Water Levels, 100 Year Event, Surface System**
Table C15	**Peak Water Levels, 2 Year Event, Underground System**
Table C16	**Peak Water Levels, 100 Year Event, Underground System**
Table C17	**Peak Flows and Velocities, 2 Year Event, Surface System**
Table C18	**Peak Flows and Velocities, 100 Year Event, Surface System**
Table C19	**Peak Flows and Velocities, 2 Year Event, Underground System**
Table C20	**Peak Flows and Velocities, 100 Year Event, Underground System**

TABLE 4.1
Lagoon Levels- 2 Year Event
Relief Drainage Options

Location	Node	Existing	Ultimate (Prior to Works)	Bund At Robert Street		Enlarged Lagoons		Bund + Enlarged Lagoons		Bund + Enlarged Lagoons + Extra Lagoon	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
Northern Lagoons											
Lagoon 73	SNLOWN01	1.71	1.84	1.79	-50	1.78	-60	1.74	-100	1.72	-120
Lagoon 60	SNLOWN06	2.19	1.98	1.91	-70	1.89	-90	1.83	-150	1.80	-180
Central Lagoon											
Lagoon 50	SNLOW01	2.23	2.15	2.07	-80	2.06	-90	1.97	-180	1.93	-220
Southern and Eastern Lagoons											
Lagoon 40	SNLOWS11	2.24	2.18	2.10	-80	2.07	-110	1.99	-190	1.95	-230
Lagoon 30	SNLOWS09	2.24	2.19	2.10	-90	2.07	-120	1.99	-200	1.95	-240
Lagoon 20- Botanic Gardens	SNLOWS06	2.38	2.39	2.40	10	2.03	-360	2.03	-360	2.03	-360
Kondari Resort Lagoon 21	SNLOWS01	1.81	1.62	1.62	0	1.75	130	1.75	130	1.75	130
Tooan Tooan Creek to West of Lagoons											
To west of Robert Street	SNLOW03	2.42	2.44	2.77	330	2.44	0	2.77	330	2.77	330
Bideford Street	SNLOW14	2.68	2.73	2.77	40	2.73	0	2.77	40	2.77	40
Denmans Camp Road	SNLOW28	2.73	2.80	2.80	0	2.80	0	2.80	0	2.80	0
Frank Street	SNLOW33	2.71	2.81	2.81	0	2.81	0	2.81	0	2.81	0

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

TABLE 4.2
Lagoon Levels- 100 Year Event
Relief Drainage Options

Location	Node	Existing	Ultimate (Prior to Works)	Bund At Robert Street		Enlarged Lagoons		Bund + Enlarged Lagoons		Bund + Enlarged Lagoons + Extra Lagoon	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
Northern Lagoons											
Lagoon 73	SNLOWN01	2.85	2.90	2.81	-90	2.84	-50	2.76	-140	2.77	-130
Lagoon 60	SNLOWN06	2.86	2.90	2.82	-80	2.85	-50	2.75	-150	2.75	-150
Central Lagoon											
Lagoon 50	SNLOW01	2.98	3.02	2.94	-80	2.98	-40	2.86	-160	2.85	-170
Southern and Eastern Lagoons											
Lagoon 40	SNLOWS11	2.99	3.02	2.94	-80	3.00	-20	2.90	-120	2.90	-120
Lagoon 30	SNLOWS09	2.99	3.03	2.94	-90	3.00	-30	2.90	-130	2.90	-130
Lagoon 20- Botanic Gardens	SNLOWS06	2.99	3.03	2.94	-90	3.00	-30	2.90	-130	2.90	-130
Kondari Resort Lagoon 21	SNLOWS01	2.98	3.03	2.94	-90	3.01	-20	2.91	-120	2.90	-130
Tooan Tooan Creek to West of Lagoons											
To west of Robert Street	SNLOW03	2.99	3.02	3.36	340	2.98	-40	3.36	340	3.36	340
Bideford Street	SNLOW14	3.10	3.16	3.35	190	3.15	-10	3.35	190	3.35	190
Denmans Camp Road	SNLOW28	3.20	3.24	3.29	50	3.24	0	3.29	50	3.29	50
Frank Street	SNLOW33	3.21	3.25	3.29	40	3.25	0	3.29	40	3.29	40

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

TABLE 4.3
Lagoon Levels- 2 Year Event
Upgrading Margaret Street and Churchill Street

Location	Node	Existing	Ultimate (Prior to Works)	Bund At Robert Street + Margaret Street		Churchill St Only		Bund at Robert Street + Churchill Street		Bund + Enlarged Lagoons + Churchill Street	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
Northern Lagoons											
Lagoon 73	SNLOWN01	1.71	1.84	1.79	-50	1.84	0	1.79	-50	1.74	-100
Lagoon 60	SNLOWN06	2.19	1.98	1.90	-80	1.97	-10	1.91	-70	1.83	-150
Central Lagoon											
Lagoon 50	SNLOW01	2.23	2.15	2.07	-80	2.15	0	2.07	-80	1.97	-180
Southern and Eastern Lagoons											
Lagoon 40	SNLOWS11	2.24	2.18	2.10	-80	2.18	0	2.10	-80	1.98	-200
Lagoon 30	SNLOWS09	2.24	2.19	2.10	-90	2.18	-10	2.10	-90	1.99	-200
Lagoon 20- Botanic Gardens	SNLOWS06	2.38	2.39	2.40	10	2.39	0	2.40	10	2.03	-360
Kondari Resort Lagoon 21	SNLOWS01	1.81	1.62	1.62	0	1.59	-30	1.59	-30	1.66	40
Tooan Tooan Creek to West of Lagoons											
To west of Robert Street	SNLOW03	2.42	2.44	2.77	330	2.44	0	2.77	330	2.77	330
Bideford Street	SNLOW14	2.68	2.73	2.77	40	2.73	0	2.77	40	2.77	40
Denmans Camp Road	SNLOW28	2.73	2.80	2.80	0	2.80	0	2.80	0	2.80	0
Frank Street	SNLOW33	2.71	2.81	2.81	0	2.81	0	2.81	0	2.81	0

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

TABLE 4.4
Lagoon Levels- 100 Year Event
Upgrading Margaret Street and Churchill Street

Location	Node	Existing	Ultimate (Prior to Works)	Bund At Robert Street + Margaret Street		Churchill St Only		Bund at Robert Street + Churchill Street		Bund + Enlarged Lagoons + Churchill Street	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
Northern Lagoons											
Lagoon 73	SNLOWN01	2.85	2.90	2.76	-140	2.87	-30	2.80	-100	2.73	-170
Lagoon 60	SNLOWN06	2.86	2.90	2.76	-140	2.88	-20	2.80	-100	2.70	-200
Central Lagoon											
Lagoon 50	SNLOW01	2.98	3.02	2.92	-100	2.94	-80	2.89	-130	2.78	-240
Southern and Eastern Lagoons											
Lagoon 40	SNLOWS11	2.99	3.02	2.92	-100	3.02	0	2.89	-130	2.80	-220
Lagoon 30	SNLOWS09	2.99	3.03	2.92	-110	3.02	-10	2.89	-140	2.80	-230
Lagoon 20- Botanic Gardens	SNLOWS06	2.99	3.03	2.92	-110	3.02	-10	2.89	-140	2.80	-230
Kondari Resort Lagoon 21	SNLOWS01	2.98	3.03	2.92	-110	3.02	-10	2.74	-290	2.80	-230
Tooan Tooan Creek to West of Lagoons											
To west of Robert Street	SNLOW03	2.99	3.02	3.36	340	2.95	-70	3.36	340	3.36	340
Bideford Street	SNLOW14	3.10	3.16	3.35	190	3.15	-10	3.35	190	3.35	190
Denmans Camp Road	SNLOW28	3.20	3.24	3.29	50	3.24	0	3.29	50	3.29	50
Frank Street	SNLOW33	3.21	3.25	3.29	40	3.25	0	3.29	40	3.29	40

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

Reducing the catchment area of the lagoons by bunding off the area to the west of Robert Street was found to provide a reduction in flood level of between 50 and 90 mm for the 2 year event and a reduction of about 80 mm for the 100 year event. Directing runoff to the west was found to increase flood levels in the west by up to 330 mm for the 2 year event and 340 mm for the 100 year event. If the bund option were to be adopted, it would be necessary to complete relief drainage works in the western part of the catchment to offset the impact of the increase (refer Section 4.3).

By itself, maximising the area of the lagoons was found to produce a flood level reduction of between 20 and 50 mm for the 100 year event. By virtue of the lesser runoff volume associated with the smaller events, maximising the lagoon storage volume had a greater effect on the 2 year event. For the 2 year event, a reduction in flood level of between 60 and 360 mm was obtained. The provision of a hydraulic link between the southern lagoons and Kondari Resort lagoon caused an increase in the amount of flow entering the lagoon and as a consequence produced an increase in lagoon level for the 2 year event of 130 mm.

Combining the bund and lagoon storage maximisation options produced flood level reductions greater than the sum of the reductions afforded by each of the options considered individually. This was attributed to the increased storage volume in the enlarged lagoons and the reduction in runoff directed to the lagoons as a result of the bunding of the catchment.

The addition of a lagoon in the Council owned area to the west of Robert Street was found to provide an additional flood level reduction of 20 to 40 mm for the 2 year event. For the 100 year event the additional reduction was of the order of zero to 10 mm. Given the relatively small improvement afforded by the addition of the lagoon, it was considered that excavation of the additional lagoon is not strictly necessary. It is suggested that excavation of the lagoon be considered in the event of enlargement options for the existing lagoons being unachievable in the future.

The augmentation of the Margaret Street system would provide an additional reduction in flood level (compared to that obtained from the bund in isolation) in the lagoons of between 20 and 60 mm for the 100 year event. However, the reasonable reduction in water level achieved in Lagoon 73 was not obtained in the other lagoons due to the head loss that occurs at the Northern crossing of Ann Street (between Lagoon 50 and Lagoon 60) and in the narrow channel to the east of Ann Street. Increasing the flow capacity of the Margaret Street system increases the volume of water draining across Ann Street. This, in turn, was found to significantly increase the head loss across Ann Street, minimising the flood level reduction obtained in the Central and Southern lagoons afforded as a result of works at Margaret Street.

A relatively shallow and narrow channel runs from Ann Street to the main part of Lagoon 60. The obstruction to flow presented by the channel accounts for the head loss calculated at Ann Street. It would only be possible to widen or deepen the channel significantly if the existing house (set at the top of a retaining wall) immediately to the north of the channel was removed.

Since the Churchill Street solution was found to provide a suitable reduction in flood level within the lagoon system the completion of additional augmentation works or the addition of pipes over the full length of the Margaret Street system was not considered further. Similarly, removal of the existing house adjacent to the channel at Ann Street was not considered to be acceptable given the superior performance of the Churchill Street solution.

The duplication of the Churchill Street system was found to produce a good reduction in flood level throughout the lagoon system. The benefit afforded by the duplication would be maximised by the addition of a bund at Robert Street and the completion of the lagoon enlargement works, in particular the link between the southern lagoons (Lagoon 30) and the Kondari Resort lagoon (Lagoon 21).

If the Robert Street bund were to be constructed together with the enlarged lagoons and the Churchill Street duplication, peak water levels in the lagoons for the 100 year event would be reduced by between 170 mm and 240 mm. The largest reduction would occur in those lagoons with the highest flood levels. It can be noted that the potential direction of runoff from the Ann and Robert Street catchments (which presently drain generally to the ocean) to the lagoons would be expected to offset the flood level reduction to some extent (refer Section 4.4.5).

The cost of completing the relief drainage options was calculated as follows:

- Margaret Street $570,000 (refer Table D4, Appendix D)
- Churchill Street $1,054,000 (refer Table D5, Appendix D)

The cost of completing the lagoon works and the Robert Street bund would need to be added to the cost of completing the Churchill Street option.

4.3 Tooan Tooan Creek West of Robert Street

Peak flood levels for the 100 year event in Tooan Tooan Creek between Robert Street and Frank Street range from RL 3.0 m AHD to RL 3.25 m AHD. Downstream of the covered section of creek to the west of Frank Street, the peak flood level for the 100 year event varies from RL 2.8 m AHD to RL 2.4 m AHD. Given that a flood level of the order of RL 2.9 m AHD is desirable for the main creek, works will be required in the reach between Robert Street and Frank Street.

As noted previously (refer Section 4.2), there is little scope for enlarging the existing channel. A reduction in flood level can therefore only be achieved by providing relief drainage systems connecting the creek to the ocean. Suitable locations for outfalls are limited to existing outfall locations (refer Section 4.1).

As a boardwalk and observation deck have been constructed over the Bideford Street outfall, it would be possible to upgrade the existing outfall and incorporate it into the boardwalk. Given the invert level of the existing outfall and existing surface levels between the outfall and the creek, it was determined that box culverts with a maximum height of 1.2 metres could be employed. For the purposes of modelling, 1.5 metre span box culverts were considered (refer **Figure 8- Bideford Street Augmentation**).

Upgrading of the existing outfall at Tavistock Street was not pursued due to the proximity of the outlet to Bideford Street. It was considered that the completion of works in Tavistock Street would provide only a minimal additional reduction in level compared to that obtained by the completion of works at Bideford Street. Given that it is more cost effective to locate relief drainage works at Bideford Street compared to Tavistock Street, no further consideration was given to the completion of works at Tavistock Street.

The only other existing outfall within the reach of interest is located at Frank Street. Due to the relatively close proximity of the creek to the ocean at Frank Street, it was considered ideal for the location of relief drainage works (refer **Figure 9- Frank Street Augmentation**). Given the invert level of the existing outfall and the relatively low surface levels between the outfall and the creek, box culverts with a height of 900 mm were adopted. 2.4 metre span culverts were ultimately adopted for the system. It was also assumed that the opportunity would be taken to remove the existing outfall immediately to the east of the Frank Street outfall and drain the low point on Charlton Esplanade via the Frank Street outfall.

Due to the shorter distance between the creek and the ocean at Frank Street, it was calculated that the cost of constructing a pipe at Frank Street would be about 40 percent cheaper than the cost of constructing a pipe of the same size at Bideford Street. It was therefore considered desirable to maximise the amount of drainage provided at Frank Street in order to minimise the overall cost of the relief drainage works.

However, during modelling it was found that the flood level reduction afforded at both Bideford Street and Frank Street was relatively localised. For example, the addition of 3/2,400 x 900 mm SLBCs at Frank Street was found to produce a maximum flood level reduction of 280 mm at Frank Street for the 100 year event. The reduction decreased to 130 mm at Bideford Street and to 14 mm upstream of Fraser Street. A solution for the creek will therefore require the construction of relief drainage at both Frank Street and Bideford Street.

The following solution was ultimately adopted for Bideford Street and Frank Street:

- Bideford Street 3/1.5 x 1.2 m SLBCs
- Frank Street 3/2.4 x 0.9 m SLBCs

For both Frank Street and Bideford Street, it was assumed that floodgates would be provided to prevent the worsening of inland flooding during large tide or storm surge events. Further, it was assumed that the new systems would not extend further on to the beach than the present outfalls. The costing (refer below) has included an allowance for the treatment of each outfall to improve its amenity.

The above sizes were found to provide an adequate reduction in flood level for the 100 year event. Although the flood level reduction was offset to some degree if a bund were to be constructed at Robert Street (refer Section 4.2), the resultant flood levels were still found to be acceptable.

The following options were modelled:

- Frank Street and Bideford Street
- Frank Street, Bideford Street, and bund at Robert Street (refer Section 4.2)
- Frank Street, Bideford Street, bund at Robert Street and flow from Macks Road and Robert Street catchments directed either to the ocean or inland to the lagoons.

The final option was modelled in order to gain an understanding of the likely ultimate flood levels to the west of Robert Street if works were completed in the Macks Road and Robert Street catchments (refer Section 4.4). Works in these catchments would result in their runoff either being directed to the ocean or to the lagoons.

Either solution together with the bund at Robert Street would reduce the volume of runoff directed to the western part of the catchment and therefore maximise the flood level reduction afforded by the relief drainage works.

The peak flood levels calculated at key points in Tooan Tooan Creek are summarized in the following tables:

- **Table 4.5** **2 Year Event, Bideford and Frank Street Relief Drainage Works**
- **Table 4.6** **100 Year Event, Bideford and Frank Street Relief Drainage Works**

Detailed results for the analysis are presented in **Appendix C- Model Results**:

Table C21 **Peak Water Levels, 2 Year Event, Surface System**
Table C22 **Peak Water Levels, 100 Year Event, Surface System**
Table C23 **Peak Water Levels, 2 Year Event, Underground System**
Table C24 **Peak Water Levels, 100 Year Event, Underground System**
Table C25 **Peak Flows and Velocities, 2 Year Event, Surface System**
Table C26 **Peak Flows and Velocities, 100 Year Event, Surface System**
Table C27 **Peak Flows and Velocities, 2 Year Event, Underground System**
Table C28 **Peak Flows and Velocities, 100 Year Event, Underground System**

TABLE 4.5
Bideford and Frank Street Relief Drainage Works
Result Summary, 2 Year Event

Location	Node	Existing	Ultimate (Prior to Works)	Bideford + Frank		Bideford + Frank + Bund at Robert Street		Bideford + Frank + Bund without Macks Rd & Robert Street	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
West of Robert Street	SNLOW03	2.42	2.44	2.36	-80	2.51	70	2.48	40
Fraser Street	SNLOW10	2.67	2.71	2.47	-240	2.47	-240	2.47	-240
Bideford Street	SNLOW14	2.68	2.73	2.32	-410	2.32	-410	2.32	-410
Tavistock Street	SNLOW19	2.68	2.74	2.55	-190	2.55	-190	2.55	-190
Denmans Camp Road	SNLOW29	2.73	2.81	2.50	-310	2.50	-310	2.50	-310
Frank Street	SNLOW33	2.71	2.81	2.14	-670	2.14	-670	2.14	-670

Note: Refer Figure 1 for location of streets
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

TABLE 4.6
Bideford and Frank Street Relief Drainage Works
Result Summary, 100 Year Event

Location	Node	Existing	Ultimate (Prior to Works)	Bideford + Frank		Bideford + Frank + Bund at Robert Street		Bideford + Frank + Bund without Macks Rd & Robert Street	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
West of Robert Street	SNLOW03	2.99	3.02	2.92	-100	3.07	50	2.96	-60
Fraser Street	SNLOW10	3.08	3.14	2.88	-260	2.99	-150	2.94	-200
Bideford Street	SNLOW14	3.10	3.16	2.89	-270	2.96	-200	2.92	-240
Tavistock Street	SNLOW19	3.13	3.19	2.94	-250	2.96	-230	2.96	-230
Denmans Camp Road	SNLOW29	3.20	3.24	2.96	-280	2.97	-270	2.97	-270
Frank Street	SNLOW 33	3.21	3.25	2.92	-330	2.92	-330	2.92	-330

Note: Refer Figure 1 for location of streets
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

Construction of the Bideford and Frank Street relief drainage works would produce a 100 year flood level of just over RL 2.9 m AHD. Constructing a bund to direct additional runoff to the western part of the Lowlands catchment would reduce the benefit provided by the Bideford and Frank Street works. However, the diversion of runoff from the Robert and Macks Street catchments (refer Section 4.4) to either the ocean or the lagoons would compensate this to some degree and would result in a design flood level approaching RL 2.9 m AHD.

Given this, it was concluded that the proposed relief drainage works at Bideford and Frank Street would provide an acceptable reduction in peak flood level, regardless of whether a bund were to be constructed at Robert Street. It can be noted that a further reduction of flood levels in this reach of Tooan Tooan Creek could not be justified as the resultant peak water levels would be less than the levels produced by storm surge.

The estimated cost to complete the works is as follows:

- Bideford Street $1,396,000 (refer Table D2)
- Frank Street $1,362,000 (refer Table D3)
- **TOTAL** **$2,758,000**

4.4 Drainage of Macks Road to Alexander Street

4.4.1 General

As shown on **Figure 3- 100 Year Inundation Plan**, the region between Macks Road and Alexander Street is relatively low lying and subject to significant inundation during flood events. Flooding is exacerbated by the lack of an overland flow path to drain this part of the catchment.

Available relief drainage measures for the region include:

- Drainage to the ocean via the upgrading of existing outfalls; and
- Drainage to the inland lagoon system.

The most significant disadvantage associated with drainage to the ocean is that the invert level of the outfall cannot be significantly lower than that of the existing outfall in order to be free draining. This limits the size of pipe which can be accommodated upstream of the outfall and the available hydraulic head.

In comparison, discharging to the lagoons offers a number of potential benefits:

- The ability to use lower invert levels to maximise pipe size and ensure adequate cover;
- Reduced tailwater levels for the drainage of internal areas (compared to the applicable design ocean level); and
- The removal of existing outfalls.

Discharging to the lagoons would add to the volume of runoff entering and therefore potentially increase the peak level reached in the lagoons. There will consequently be a need to complete additional relief drainage works to provide acceptable flood levels within the lagoons (refer Section 4.4.5).

The relief drainage works proposed for Macks Road, Robert Street, and Ann Street are described in the following sections. In all cases, systems were initially designed to provide acceptable performance for major event flooding. Due to the lack of an overland flow path for the drainage of major event flows, the size of the underground drainage system needs to be larger than that required to convey the minor event flow. The drainage of major event flows is achieved by a combination of storing water above ground by limiting the rate at which flow enters the underground drainage system and increasing the size of the underground drainage system.

When sizing the pipes required for the area, it was recognised that the cost of providing a system capable of minimising water depths to within acceptable limits for the major flood event would be significantly greater than the cost of the system necessary to convey the minor event flow. Although greater flood depths would occur during major events if a minor event system standard were to be adopted, provided the resultant depths were still within acceptable limits, the minor event drainage solution would be the most cost effective approach for the completion of relief drainage works.

Such a system would provide the required immunity to flooding for minor events and minimise the incidence of nuisance flooding. While flooding would occur for major events, such flooding would occur relatively infrequently. Flood immunity could be achieved within properties by either filling the properties as they are redeveloped or by adopting habitable floor levels greater than the major event flood level.

Given the above, in a number of cases two drainage solutions were developed:

- Relief drainage works to provide adequate minor drainage and some improvement in major event flooding; and
- Relief drainage works to provide adequate minor and major event drainage.

For cases where the difference between the two solutions was relatively slight, only the major drainage solution is presented in this report.

For the analysis, it was assumed that no relief drainage works would be completed within the lagoons. This led to the conservative overestimation of tailwater levels in the lagoons.

The reduction in flood level afforded by each relief drainage scheme at key points is summarised in the following sections. Comprehensive results are presented in **Appendix C- Model Results** in the following tables:

- *Macks Road to Ocean (refer Section 4.4.2)*

Table C29	**Peak Water Levels, 2 and 100 Year Events, Surface System**
Table C30	**Peak Water Levels, 2 and 100 Year Events, Underground System**
Table C31	**Peak Flows and Velocities, 2 and 100 Year Events, Surface System**
Table C32	**Peak Flows and Velocities, 2 and 100 Year Events, Underground System**

- *Robert Street (refer Section 4.4.3)*

Table C33	**Peak Water Levels, 2 Year Event, Surface System**
Table C34	**Peak Water Levels, 100 Year Event, Surface System**
Table C35	**Peak Water Levels, 2 Year Event, Underground System**
Table C36	**Peak Water Levels, 100 Year Event, Underground System**
Table C37	**Peak Flows and Velocities, 2 Year Event, Surface System**
Table C38	**Peak Flows and Velocities, 100 Year Event, Surface System**
Table C39	**Peak Flows and Velocities, 2 Year Event, Underground System**
Table C40	**Peak Flows and Velocities, 100 Year Event, Underground System**

- *Ann Street (refer Section 4.4.4)*

Table C41	**Peak Water Levels, 2 Year Event, Surface System**
Table C42	**Peak Water Levels, 100 Year Event, Surface System**
Table C43	**Peak Water Levels, 2 Year Event, Underground System**
Table C44	**Peak Water Levels, 100 Year Event, Underground System**
Table C45	**Peak Flows and Velocities, 2 Year Event, Surface System**
Table C46	**Peak Flows and Velocities, 100 Year Event, Surface System**
Table C47	**Peak Flows and Velocities, 2 Year Event, Underground System**
Table C48	**Peak Flows and Velocities, 100 Year Event, Underground System**

- *Combined Effect on Lagoon Levels (refer Section 4.4.5)*

Table C49	**Peak Water Levels, 2 Year Event, Surface System**
Table C50	**Peak Water Levels, 100 Year Event, Surface System**
Table C51	**Peak Water Levels, 2 Year Event, Underground System**
Table C52	**Peak Water Levels, 100 Year Event, Underground System**
Table C53	**Peak Flows and Velocities, 2 Year Event, Surface System**
Table C54	**Peak Flows and Velocities, 100 Year Event, Surface System**
Table C55	**Peak Flows and Velocities, 2 Year Event, Underground System**
Table C56	**Peak Flows and Velocities, 100 Year Event, Underground System**

4.4.2 Macks Road

The works required to drain the Macks Road subcatchment to the ocean (Option A) are shown on **Figure 11- Macks Road Relief Drainage Works**. Resultant peak flood levels at points along Macks Road are tabulated in **Table 4.7- Macks Road Relief Drainage Works, Option A- Drain to Ocean**.

TABLE 4.7
Macks Road Relief Drainage Works
Option A- Drain To Ocean
Result Summary

Location	Model Node	Ground Level	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
		(m AHD)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations on Macks Road										
Charlton Esp.	SNMACKS05	3.61	3.70	90	3.68	70	3.77	160	3.72	110
Cypress Street	SNMACKS04	3.22	3.46	240	3.32	100	3.63	410	3.56	340
Ocean Street	SNMACKS03	3.27	3.44	170	3.33	60	3.62	350	3.56	290
View Street	SNMACKS02	3.38	3.43	50	3.40	20	3.57	190	3.52	140
Truro Street	SNMACKS01	3.22	3.29	70	3.28	60	3.40	180	3.36	140

Note: Refer Figure 11 for pipe dimensions and locations.
Locations where depth of water is greater than desirable shaded

The low point at the intersection of Macks Road and Cypress Street governed the sizing of the drainage system. The flood depth of 340 mm at Cypress Street for the 100 year event was considered to be within acceptable limits. Reducing the flood depth further would have required significant additional works that could not be justified given the acceptable result obtained for the 2 year event.

Consideration was also given to draining Macks Road to the lagoons in order to allow the removal of the existing outfall. For this analysis, it was assumed that the Macks Road system would join the Robert Street system and discharge to the lagoons via an open channel in the Robert Street road reserve. The results of this analysis are presented in Section 4.4.3 (Option C).

4.4.3 Robert Street

The following drainage solutions were developed for Robert Street:

- Option A- Drain to Ocean (refer **Figure 12- Robert Street Relief Drainage Works, Option A- Drain to Ocean**)
- Option B1- Drain to Lagoons, Minor Event solution (refer **Figure 13- Robert Street Relief Drainage Works, Option B1- Drain to Lagoons, Minor Event Solution**)
- Option B2- Drain to Lagoons, Major Event solution (refer **Figure 14- Robert Street Relief Drainage Works, Option B2- Drain to Lagoons, Major Event Solution**)
- Option C- Drain to Lagoons, Minor Event solution (refer **Figure 15- Robert Street Relief Drainage Works, Option C- Drain Robert Street and Macks Road to Lagoons**)

In all cases, it was considered preferable to remove the existing outfall at Eric Street, with the low point in Eric Street between Charlton Esplanade and Cypress Street drained via Robert Street.

The peak flood levels calculated within the system are summarised in the following tables:

- **Table 4.8- Option A- Drain to Ocean**
- **Table 4.9- Option B1- Drain to Lagoons, Minor Event Solution**
- **Table 4.10- Option B2- Drain to Lagoons, Major Event Solution**
- **Table 4.11- Option C- Drain Robert Street and Macks Road to Lagoons**

TABLE 4.8
Robert Street Relief Drainage Works
Option A- Drain To Ocean
Result Summary

Location	Model Node	Ground Level	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
		(m AHD)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations on Robert Street										
Charlton Esp.	SNROB08	4.24	4.27	30	4.27	30	4.30	60	4.29	50
Cypress Street	SNROB06	3.39	3.48	90	3.46	70	3.62	230	3.54	150
Ocean Street	SNROB05	3.08	3.44	360	3.19	110	3.62	540	3.41	330
View Street	SNROB04	3.21	3.44	230	3.26	50	3.61	400	3.41	200
Truro Street	SNROB03	3.47	3.49	20	3.49	20	3.59	120	3.50	30
Other Locations										
Eric St between Charlton Esp & Cypress St	SNERIC05	3.24	3.38	140	3.35	110	3.66	420	3.49	250
Cnr Eric St and Truro St	SNERIC01	3.20	3.36	160	3.31	110	3.46	260	3.44	240

Note: *Refer Figure 12 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded

The low point at the intersection of Robert Street and Ocean Street governed the sizing of the drainage system for Option A. The flood depth of 330 mm for the 100 year event was considered to be within acceptable limits. Reducing the flood depth further would have required significant additional works that could not be justified given the acceptable result obtained for the 2 year event.

TABLE 4.9
Robert Street Relief Drainage Works
Option B1- Drain To Lagoons
Minor Event Solution- Result Summary

Location	Model Node	Ground Level	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
		(m AHD)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations on Robert Street										
Charlton Esp.	SNROB08	4.24	4.27	30	4.26	20	4.30	60	4.28	40
Cypress Street	SNROB06	3.39	3.48	90	3.46	70	3.62	230	3.56	170
Ocean Street	SNROB05	3.08	3.44	360	3.18	100	3.62	540	3.53	450
View Street	SNROB04	3.21	3.44	230	3.26	50	3.61	400	3.52	310
Truro Street	SNROB03	3.47	3.49	20	3.49	20	3.59	120	3.52	50
Other Locations										
Eric St between Charlton Esp & Cypress St	SNERIC05	3.24	3.38	140	3.35	110	3.66	420	3.61	370
Cnr Eric St and Truro St	SNERIC01	3.20	3.36	160	3.31	110	3.46	260	3.40	200

Note: *Refer Figure 13 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded

TABLE 4.10
Robert Street Relief Drainage Works
Option B2- Drain To Lagoons
Major Event Solution- Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
			Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations on Robert Street										
Charlton Esp.	SNROB08	4.24	4.27	30	4.26	20	4.30	60	4.29	50
Cypress Street	SNROB06	3.39	3.48	90	3.46	70	3.62	230	3.55	160
Ocean Street	SNROB05	3.08	3.44	360	3.18	100	3.62	540	3.34	260
View Street	SNROB04	3.21	3.44	230	3.26	50	3.61	400	3.34	130
Truro Street	SNROB03	3.47	3.49	20	3.49	20	3.59	120	3.50	30
Other Locations										
Eric St between Charlton Esp & Cypress St	SNERIC05	3.24	3.38	140	3.35	110	3.66	420	3.52	280
Cnr Eric St and Truro St	SNERIC01	3.20	3.36	160	3.31	110	3.46	260	3.40	200

Note: *Refer Figure 14 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded

With Option B1 (minor event solution), relatively large depths of flooding were obtained at Ocean Street, View Street and in Eric Street. The resultant levels were still considered to be acceptable given the relatively rare nature of the 100 year event. However, filling of properties in these areas would be required to prevent significant flood damage from occurring.

The increased pipe sizes provided for the Option B2 (major event) solution provide flood levels generally within QUDM standards at all locations except Eric Street, where the calculated depth is within 10 mm of the QUDM standard and therefore considered to be within reasonable limits.

Option C is an extension of Option B, with the Macks Road system added. A peak flood depth at the intersection of Macks Road and Cypress Street similar to that obtained for Option A was calculated. As for Option A, the resultant depth was considered to be within acceptable limits. Similarly, the level in Eric Street near Charlton Esplanade was also considered to be acceptable.

It should be noted that the flood depth predicted at the intersection of Ocean Street and Robert Street is 230 mm for Option C and 260 mm for Option B2. This result was attributed to the fact that Option B2 was modelled assuming no drainage augmentation works in Macks Road. As a result, flow drained from Macks Road to Robert Street, producing increased levels in Robert Street. With the addition of the Macks Road relief drainage to the model in Option C, the tendency of flow to transfer from Macks Road to Robert Street was reduced, resulting in a smaller depth of flooding at Robert Street.

TABLE 4.11
Robert Street Relief Drainage Works
Option C- Drain Robert St and Macks Rd To Lagoons
Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
			Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations on Robert Street										
Charlton Esp.	SNROB08	4.24	4.27	30	4.27	30	4.30	60	4.29	50
Cypress Street	SNROB06	3.39	3.48	90	3.46	70	3.62	230	3.53	140
Ocean Street	SNROB05	3.08	3.44	360	3.17	90	3.62	540	3.31	230
View Street	SNROB04	3.21	3.44	230	3.26	50	3.61	400	3.31	100
Truro Street	SNROB03	3.47	3.49	20	3.49	20	3.59	120	3.50	30
Other Locations										
Eric St between Charlton Esp & Cypress St	SNERIC05	3.24	3.38	140	3.35	110	3.66	420	3.53	290
Cnr Eric St and Truro St	SNERIC01	3.20	3.36	160	3.31	110	3.46	260	3.40	200
Locations on Macks Road										
Charlton Esp.	SNMACKS05	3.61	3.70	90	3.68	70	3.77	160	3.73	120
Cypress Street	SNMACKS04	3.22	3.46	240	3.32	100	3.63	410	3.55	330
Ocean Street	SNMACKS03	3.27	3.44	170	3.31	40	3.62	350	3.49	220
View Street	SNMACKS02	3.38	3.43	50	3.40	20	3.57	190	3.45	70
Truro Street	SNMACKS01	3.22	3.29	70	3.28	60	3.40	180	3.31	90

Note: Refer Figure 15 for pipe dimensions and locations.
Locations where depth of water is greater than desirable shaded

4.4.4 Ann St

The following drainage solutions were developed for Ann Street:

- Option A1- Drain to Ocean, Minor Event Solution (refer **Figure 16- Ann Street Relief Drainage Works, Option A1- Drain to Ocean, Minor Event Solution**)
- Option A2- Drain to Ocean, Major Event Solution (refer **Figure 17- Ann Street Relief Drainage Works, Option A1- Drain to Ocean, Major Event Solution**)
- Option B1- Drain to Lagoons, Minor Event Solution (refer **Figure 18- Ann Street Relief Drainage Works, Option B1- Drain to Lagoons, Minor Event Solution**)
- Option B2- Drain to Lagoons, Minor Event Solution (refer **Figure 19- Ann Street Relief Drainage Works, Option B2- Drain to Lagoons, Major Event Solution**)

As Ann Street has only recently been resurfaced, for Option A the extent of works required in Ann Street was minimised by using the existing Crown Street outfall for the drainage of the majority of Ann Street. To reduce flooding in areas serviced by the existing drainage system, the catchment area of the existing Ann Street system was reduced until the existing system downstream of Cypress Street functioned adequately.

For Option B, it was not possible to minimise impacts on Ann Street as there would be no reasonable way to avoid the placement of trunk drainage within Ann Street.

In all cases, it was assumed that the existing outfall between Crown Street and Alexander Street was removed and the water collected at the sag directed to the Crown Street system. Drainage of Ann Street to the lagoons (Option B) would allow the removal of three existing outfalls.

The peak flood levels calculated within the system are summarised in the following tables:

- **Table 4.12- Option A1- Drain to Ocean, Minor Event Solution**
- **Table 4.13- Option A2- Drain to Ocean, Major Event Solution**
- **Table 4.14- Option B1- Drain to Lagoons, Minor Event Solution**
- **Table 4.15- Option B2- Drain to Lagoons, Major Event Solution**

For all options modelled, the flood depths at the Pebble Street cul de sac and the low point in Cunningham Street governed the sizing of pipes within the system. In the case of Option A (drain to ocean), drainage of these low points was made difficult by the small cover available for pipes and the distance over which drainage is required.

As a result of these factors, a peak depth at Pebble Street of 580 mm was obtained for the 100 year event for the minor event solution. The addition of pipes to the system provided relatively minimal improvement, with the depth of flooding reducing to 440 mm for the major event solution. The addition of further pipes was found to produce increasingly smaller reductions in flood depth. Increasing the number of pipes could therefore not be justified. For both the minor and major event solutions some filling of lots will be required to provide flood immunity. However, either of the solutions would eliminate nuisance flooding and would therefore be worthwhile.

For Option B, the ability to set a relatively low invert level and use larger diameter pipes together with the reduced tailwater associated with the lagoons resulted in a significantly superior solution being obtained for both minor and major event solutions. For the major event solution, the calculated level at Pebble Street was within 10 mm of the desirable limit according to QUDM.

TABLE 4.12
Ann Street Relief Drainage Works
Option A1- Drain To Ocean
Minor Event Solution- Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
			Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations On Ann Street										
Charlton Esp	SNANNN09	3.36	3.47	110	3.45	90	3.69	330	3.56	200
Sag between Charlton Esp & Cypress St	SNANNN08	3.35	3.47	120	3.47	120	3.69	340	3.56	210
Cypress Street	SNANNN07	3.91	3.94	30	3.94	30	3.97	60	3.97	60
Cunningham St	SNANNN06	3.25	3.53	280	3.33	80	3.82	570	3.61	360
Keys Avenue	SNANNN04	3.45	3.54	90	3.49	35	3.82	370	3.61	160
Truro Street	SNANNN03	3.49	3.60	110	3.57	80	3.79	300	3.63	140
Locations off Ann Street										
Cypress St at Witt Street	SNWIL01	3.34	3.54	200	3.46	120	3.70	360	3.64	300
Low point in Cunningham St	SNBRANN02	3.15	3.53	380	3.27	120	3.82	670	3.60	450
Keys Av near Ann Street	SNKEYS03	3.28	3.54	260	3.37	90	3.82	540	3.61	330
Keys Av at Pebble Court	SNKEYS01	3.29	3.63	340	3.33	40	3.83	540	3.71	420
Pebble Court	SNDEB01	3.13	3.63	500	3.22	90	3.83	700	3.71	580
Locations On Crown and Brown Streets										
Charlton Esp.	SNCROWN01	3.67	3.73	60	3.73	60	3.80	130	3.76	90
Cypress Street at Crown Street	SNCROWN02	3.29	3.48	190	3.41	120	3.69	400	3.56	270
Cypress Street at Brown Street	SNBROWN03	3.60	3.66	60	3.65	50	3.82	220	3.70	100
Cunningham St	SNBROWN02	3.31	3.53	220	3.39	80	3.82	510	3.60	290
Christine Ave	SNBROWN01	3.46	3.55	90	3.53	70	3.82	360	3.60	140
Locations off Crown and Brown Streets										
Charlton Esp bet. Crown St & Alexander Sr	SNALCR01	3.19	3.40	210	3.32	125	3.69	500	3.55	360
Alexander St at Christine Av	SNALEXN04	2.97	3.23	260	3.09	120	3.62	650	3.19	220
Low point in Truro St	SNTRURO01	3.28	3.48	200	3.42	140	3.87	590	3.79	510

Note: *Refer Figure 16 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded

TABLE 4.13
Ann Street Relief Drainage Works
Option A2- Drain To Ocean
Major Event Solution- Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
			Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations On Ann Street										
Charlton Esp	SNANNN09	3.36	3.47	110	3.45	90	3.69	330	3.55	190
Sag between Charlton Esp & Cypress St	SNANNN08	3.35	3.47	120	3.47	120	3.69	340	3.56	210
Cypress Street	SNANNN07	3.91	3.94	30	3.94	30	3.97	60	3.97	60
Cunningham St	SNANNN06	3.25	3.53	280	3.33	80	3.82	570	3.49	240
Keys Avenue	SNANNN04	3.45	3.54	90	3.49	35	3.82	370	3.55	100
Truro Street	SNANNN03	3.49	3.60	110	3.57	80	3.79	300	3.63	140
Locations off Ann Street										
Cypress St at Witt Street	SNWIL01	3.34	3.54	200	3.46	120	3.70	360	3.64	300
Low point in Cunningham St	SNBRANN02	3.15	3.53	380	3.27	120	3.82	670	3.48	330
Keys Av near Ann Street	SNKEYS03	3.28	3.54	260	3.37	90	3.82	540	3.55	270
Keys Av at Pebble Court	SNKEYS01	3.29	3.63	340	3.33	40	3.83	540	3.57	280
Pebble Court	SNDEB01	3.13	3.63	500	3.22	90	3.83	700	3.57	440
Locations On Crown and Brown Streets										
Charlton Esp.	SNCROWN01	3.67	3.73	60	3.73	60	3.80	130	3.76	90
Cypress Street at Crown Street	SNCROWN02	3.29	3.48	190	3.41	120	3.69	400	3.54	250
Cypress Street at Brown Street	SNBROWN03	3.60	3.66	60	3.65	50	3.82	220	3.69	90
Cunningham St	SNBROWN02	3.31	3.53	220	3.39	80	3.82	510	3.47	160
Christine Ave	SNBROWN01	3.46	3.55	90	3.53	70	3.82	360	3.57	110
Locations off Crown and Brown Streets										
Charlton Esp bet. Crown St & Alexander Sr	SNALCR01	3.19	3.40	210	3.32	125	3.69	500	3.50	310
Alexander St at Christine Av	SNALEXN04	2.97	3.23	260	3.09	120	3.62	650	3.19	220
Low point in Truro St	SNTRURO01	3.28	3.48	200	3.42	140	3.87	590	3.59	310

Note: Refer Figure 17 for pipe dimensions and locations.
Locations where depth of water is greater than desirable shaded

TABLE 4.14
Ann Street Relief Drainage Works
Option B1- Drain To Lagoons
Minor Event Solution- Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
			Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations On Ann Street										
Charlton Esp	SNANNN09	3.36	3.47	110	3.43	70	3.69	330	3.65	290
Sag between Charlton Esp & Cypress St	SNANNN08	3.35	3.47	120	3.44	90	3.69	340	3.65	300
Cypress Street	SNANNN07	3.91	3.94	30	3.94	30	3.97	60	3.96	50
Cunningham St	SNANNN06	3.25	3.53	280	3.33	80	3.82	570	3.55	300
Keys Avenue	SNANNN04	3.45	3.54	90	3.48	30	3.82	370	3.55	100
Truro Street	SNANNN03	3.49	3.60	110	3.57	80	3.79	300	3.62	130
Locations off Ann Street										
Cypress St at Witt Street	SNWIL01	3.34	3.54	200	3.41	70	3.70	360	3.65	310
Low point in Cunningham St	SNBRANN02	3.15	3.53	380	3.27	120	3.82	670	3.55	400
Keys Av near Ann Street	SNKEYS03	3.28	3.54	260	3.37	90	3.82	540	3.55	270
Keys Av at Pebble Court	SNKEYS01	3.29	3.63	340	3.33	40	3.83	540	3.63	340
Pebble Court	SNDEB01	3.13	3.63	500	3.22	90	3.83	700	3.63	500
Anembo Drive near Ann St	SNALAN04	2.75	2.87	120	2.87	120	3.11	360	2.98	230
Anembo Drive at Rosalind Court	SNALAN03	3.21	3.25	40	3.26	50	3.30	90	3.29	80
Locations On Crown and Brown Streets										
Charlton Esp.	SNCROWN01	3.67	3.73	60	3.72	50	3.80	130	3.76	90
Cypress Street at Crown Street	SNCROWN02	3.29	3.48	190	3.40	110	3.69	400	3.66	370
Cypress Street at Brown Street	SNBROWN03	3.60	3.66	60	3.65	50	3.82	220	3.70	100
Cunningham St	SNBROWN02	3.31	3.53	220	3.39	80	3.82	510	3.55	240
Christine Ave	SNBROWN01	3.46	3.55	90	3.53	70	3.82	360	3.57	110
Locations off Crown and Brown Streets										
Charlton Esp bet. Crown St & Alexander Sr	SNALCR01	3.19	3.40	210	3.32	130	3.69	500	3.66	470
Alexander St at Christine Av	SNALEXN04	2.97	3.23	260	3.06	90	3.62	650	3.24	270
Low point in Truro St	SNTRURO01	3.28	3.48	200	3.42	140	3.87	590	3.79	510

Note: *Refer Figure 18 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded

TABLE 4.15
Ann Street Relief Drainage Works
Option B2- Drain To Lagoons
Major Event Solution- Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event Ultimate Case Max Water Level (m AHD)	2 Year Event Ultimate Case Max Water Depth (mm)	2 Year Event With Relief Works Max Water Level (m AHD)	2 Year Event With Relief Works Max Water Depth (mm)	100 Year Event Ultimate Case Max Water Level (m AHD)	100 Year Event Ultimate Case Max Water Depth (mm)	100 Year Event With Relief Works Max Water Level (m AHD)	100 Year Event With Relief Works Max Water Depth (mm)
Locations On Ann Street										
Charlton Esp	SNANNN09	3.36	3.47	110	3.43	70	3.69	330	3.52	160
Sag between Charlton Esp & Cypress St	SNANNN08	3.35	3.47	120	3.44	90	3.69	340	3.51	160
Cypress Street	SNANNN07	3.91	3.94	30	3.94	30	3.97	60	3.96	50
Cunningham St	SNANNN06	3.25	3.53	280	3.33	80	3.82	570	3.40	150
Keys Avenue	SNANNN04	3.45	3.54	90	3.48	30	3.82	370	3.52	70
Truro Street	SNANNN03	3.49	3.60	110	3.57	80	3.79	300	3.62	130
Locations off Ann Street										
Cypress St at Witt Street	SNWIL01	3.34	3.54	200	3.41	70	3.70	360	3.56	220
Low point in Cunningham St	SNBRANN02	3.15	3.53	380	3.27	120	3.82	670	3.42	270
Keys Av near Ann Street	SNKEYS03	3.28	3.54	260	3.37	90	3.82	540	3.44	160
Keys Av at Pebble Court	SNKEYS01	3.29	3.63	340	3.33	40	3.83	540	3.41	120
Pebble Court	SNDEB01	3.13	3.63	500	3.22	90	3.83	700	3.41	280
Anembo Drive near Ann St	SNALAN04	2.75	2.87	120	2.87	120	3.11	360	2.98	230
Anembo Drive at Rosalind Court	SNALAN03	3.21	3.25	40	3.26	50	3.30	90	3.29	80
Locations On Crown and Brown Streets										
Charlton Esp.	SNCROWN01	3.67	3.73	60	3.72	50	3.80	130	3.76	90
Cypress Street at Crown Street	SNCROWN02	3.29	3.48	190	3.40	110	3.69	400	3.52	230
Cypress Street at Brown Street	SNBROWN03	3.60	3.66	60	3.65	50	3.82	220	3.69	90
Cunningham St	SNBROWN02	3.31	3.53	220	3.39	80	3.82	510	3.46	150
Christine Ave	SNBROWN01	3.46	3.55	90	3.53	70	3.82	360	3.57	110
Locations off Crown and Brown Streets										
Charlton Esp bet. Crown St & Alexander Sr	SNALCR01	3.19	3.40	210	3.32	130	3.69	500	3.48	290
Alexander St at Christine Av	SNALEXN04	2.97	3.23	260	3.06	90	3.62	650	3.14	170
Low point in Truro St	SNTRURO01	3.28	3.48	200	3.42	140	3.87	590	3.59	310

Note: Refer Figure 19 for pipe dimensions and locations.
Locations where depth of water is greater than desirable shaded

4.4.5 Drainage to Lagoons- Impact on Lagoon Levels

The drainage of Macks Road, Robert Street, and Ann Street to the lagoons is attractive for a number of reasons (refer Section 4.4.1). However, the resultant increase in the volume of runoff discharged to the lagoons could increase the peak level reached in the lagoons. If it is decided to discharge to the lagoons, certain relief drainage works will be required to ameliorate the impact of the increased volume of runoff.

For the analysis, it was assumed that the runoff from Macks Road, Robert Street, and Ann Street would be directed to the lagoons. The following mitigation scenarios were considered:

- Bund at Robert Street
- Enlarged lagoons
- Churchill Street
- Bund + enlarged lagoons + Churchill Street

The results of the analysis are presented in **Table 4.16- Lagoon Levels- 2 Year Event, Macks Road, Robert Street and Ann Street Runoff directed to Lagoons**, and **Table 4.17- Lagoon Levels- 100 Year Event, Macks Road, Robert Street and Ann Street Runoff directed to Lagoons**.

TABLE 4.16
Lagoon Levels- 2 Year Event
Macks Road, Robert Street and Ann Street Runoff Directed to Lagoons

Location	Node	Existing	Ultimate (Prior to Works)	Enlarged Lagoons		Bund At Robert Street		Churchill St Only		Bund + Enlarged Lagoons + Churchill Street	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
Northern Lagoons											
Lagoon 73	SNLOWN01	1.71	1.84	1.82	-20	1.84	0	1.87	30	1.79	-50
Lagoon 60	SNLOWN06	2.19	1.98	1.93	-50	1.96	-20	2.02	40	1.88	-100
Central Lagoon											
Lagoon 50	SNLOW01	2.23	2.15	2.07	-80	2.09	-60	2.16	10	2.00	-150
Southern and Eastern Lagoons											
Lagoon 40	SNLOWS11	2.24	2.18	2.08	-100	2.12	-60	2.19	10	2.02	-160
Lagoon 30	SNLOWS09	2.24	2.19	2.08	-110	2.12	-70	2.19	0	2.02	-170
Lagoon 20- Botanic Gardens	SNLOWS06	2.38	2.39	2.03	-360	2.40	10	2.39	0	2.03	-360
Kondari Resort Lagoon 21	SNLOWS01	1.81	1.62	1.75	130	1.62	0	1.59	-30	1.66	40

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

TABLE 4.17
Lagoon Levels- 100 Year Event
Macks Road, Robert Street and Ann Street Runoff Directed to Lagoons

Location	Node	Existing	Ultimate (Prior to Works)	Enlarged Lagoons		Bund At Robert Street		Churchill St Only		Bund + Enlarged Lagoons + Churchill Street	
		(mAHD)	(mAHD)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)	Peak Level (mAHD)	Diff. (mm)
Northern Lagoons											
Lagoon 73	SNLOWN01	2.85	2.90	2.91	10	2.89	-10	2.89	-10	2.77	-130
Lagoon 60	SNLOWN06	2.86	2.90	2.91	10	2.89	0	2.89	-10	2.76	-140
Central Lagoon											
Lagoon 50	SNLOW01	2.98	3.02	3.01	-10	2.98	-40	2.95	-70	2.81	-210
Southern and Eastern Lagoons											
Lagoon 40	SNLOWS11	2.99	3.02	3.03	10	2.98	-40	2.95	-70	2.81	-210
Lagoon 30	SNLOWS09	2.99	3.03	3.03	0	2.99	-40	2.95	-80	2.81	-220
Lagoon 20- Botanic Gardens	SNLOWS06	2.99	3.03	3.03	0	2.98	-50	2.94	-90	2.80	-230
Kondari Resort Lagoon 21	SNLOWS01	2.98	3.03	3.03	0	2.98	-50	2.91	-120	2.80	-230

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

The results presented in Tables 4.16 and 4.17 indicate that as long as the Churchill Street duplication is undertaken the redirection of runoff from the Macks Road, Robert Street and Ann Street catchments to the lagoons would not result in an unacceptable increase in lagoon flood level.

When comparing the reduction in flood level afforded by the scenario of bund + enlarged lagoons + Churchill Street with that obtained with the existing Macks/Robert/Ann Street drainage in place (refer Table 4.4), it was noted that the flood level reduction predicted for the southern lagoons was similar for the case of redirecting runoff to the lagoons, despite the increased volume of runoff directed to the lagoons.

Following a review of the results, it was noted that the relief drainage works would reduce the response time of the Macks Road to Alexander Street area. Further, the area would drain over a shorter period following the completion of the works. It was concluded that the faster response allowed the lagoons to fill and discharge more rapidly at an earlier point in time, leading to a similar peak level in the lagoons despite the increase in runoff volume.

Consequently, it was concluded that the Macks Road to Alexander Street area could be successfully drained to the lagoons and a number of existing outfalls removed provided the Churchill Street outfall is duplicated. Six existing outfalls could be removed, leaving a 1.8 km length of foreshore between the Fraser Street and Margaret Street free from obstruction.

4.4.6 Cost of Works

The estimated cost of the Macks Road, Robert Street, and Ann Street works is presented in **Table 4.18- Estimated Construction Cost, Macks Road, Robert Street, Ann Street**.

TABLE 4.18
Estimated Construction Cost
Macks Road, Robert Street, Ann Street

Option	Cost ($)
Macks Road, Option A- Drain to Ocean	430,200
Robert Street, Option A- Drain to Ocean	1,077,000
Robert Street, Option B1- Drain to Lagoon, Minor Event Solution	750,400
Robert Street, Option B2- Drain to Lagoon, Major Event Solution	925,400
Robert Street and Macks Road, Option C- Drain to Lagoon	1,356,000
Ann Street, Option A1- Drain to Ocean, Minor Event Solution	2,003,000
Ann Street, Option A2- Drain to Ocean, Major Event Solution	2,580,000
Ann Street, Option B1- Drain to Lagoon, Minor Event Solution	2,567,000
Ann Street, Option B2- Drain to Lagoon, Major Event Solution	3,987,000

Drainage to the ocean would cost between $3,510,000 and $4,090,000 and would allow the removal of two existing outfalls.

Drainage to the lagoons would cost between $3,923,000 and $5,343,000 (plus the proportion of the cost of those mitigation works required to compensate for draining runoff to the lagoons) and would allow the removal of six existing outfalls.

Given the relatively high incremental costs associated with providing major event solutions, the most cost effective approach for Council would be to adopt the minor event solution and require new developments to either fill their lots or locate habitable floor levels above the calculated design flood level.

4.5 Other Drainage Problem Areas

4.5.1 Elizabeth Street

Four problem drainage areas were identified in the south eastern part of the catchment near Elizabeth Street and Dayman Street (refer **Figure 20- Elizabeth Street Relief Drainage Works**):

- The open area to the east of the old rail corridor (refer also **Figure 3- 100 Year Event Inundation Plan**), where ponded water causes yard flooding and produces high tailwater levels for pipe drainage
- Miller Street
- Ross Street
- Owen Street/ Boat Harbour Drive

It may be possible to excavate a lagoon within the open area to the east of the rail corridor. If a lagoon were to be constructed within the area, due to the relatively high existing ground levels downstream of the open area it would not be possible to achieve a standing water level of RL 1.5 m AHD in the lagoon to match the standing water level in the main Lowlands Lagoons.

Based on existing ground levels, it was considered that a water level of RL 2.7 m AHD could be readily achieved within the lagoon. For the analysis, it was assumed that the area was excavated to a level of RL 2.7 m AHD. As any area below this level would not contribute to flood storage, it was not necessary to model any greater amount of excavation. This approach would allow the creation of a generally dry basin with a nominal lagoon added for visual amenity if necessary. The dry area could be used for park land or recreation purposes for the majority of the time. Further, the calculated depth of flooding in the open area (refer below) would not be excessive.

The creation of the retention basin would provide a significant reduction in the tailwater level applicable to the undergound drainage systems draining to the open area and therefore the size of any relief drainage works required. The pipe works necessary to improve the drainage of the remaining drainage problem areas, provided the lagoon is constructed, are shown on **Figure 20- Elizabeth Street Relief Drainage Works**.

The reduction in flood level afforded by the relief drainage works is summarised in **Table 4.19- Elizabeth Street Relief Drainage Works**. Comprehensive results are presented in **Appendix C- Model Results** in the following tables:

Table C57	**Peak Water Levels, 2 and 100 Year Events, Surface System**
Table C58	**Peak Water Levels, 2 and 100 Year Events, Underground System**
Table C59	**Peak Flows and Velocities, 2 and 100 Year Events, Surface System**
Table C60	**Peak Flows and Velocities, 2 and 100 Year Events, Underground System**

The estimated cost to complete the Elizabeth Street works is $1,278,000. However, it should be noted that earthworks comprise about 25 percent of this total.

The impact of the additional flood storage created by the excavation of the lagoon was modelled. The calculated peak levels in the lagoons are presented in **Table 4.20- Lagoon Levels- Addition of Elizabeth Street Lagoon**.

The results indicate that the lagoon would provide a small reduction in the peak flood level reached in the main lagoon system for the 100 year event of the order of 10 to 20 mm and a reasonable reduction in lagoon peak flood level for the 2 year event. Although the main benefit associated with the excavation of a lagoon would be the elimination of backyard flooding in the vicinity of the lagoon, the flood storage volume could be useful if it proves difficult to achieve the enlarged lagoon areas (refer Section 4.2) in the future.

If a lagoon or dry basin were not to be constructed, it would be necessary to provide additional underground drainage for the Ross Street and Boat Harbour Drive outlets (refer **Figure 20- Elizabeth Street Relief Drainage Works**). Due to the relatively low level of Miller Street, it would not be possible to achieve a reduction in flood level as the tailwater level immediately downstream of Miller Street is sufficient to cause flooding of the street.

TABLE 4.19
Elizabeth Street Relief Drainage Works
Result Summary

Location	Model Node	Ground Level	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
		(m AHD)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Miller Street and Drain Downstream										
Smith Street	SNMILL03	4.01	4.20	190	4.14	130	4.36	350	4.24	230
Old Rail Corridor	SNMILL04	3.92	4.02	100	3.99	70	4.34	420	4.03	110
Florence St	SNMILL05	3.80	4.02	220	3.90	100	4.34	540	4.08	280
D/Stream end of rail corridor	SNLOWSS02	2.78	3.64	N/A	3.21	N/A	4.17	N/A	3.64	N/A
Ross Street										
Whittaker Cl.	SNROSS02	4.30	4.50	200	4.39	90	4.64	340	4.51	210
Boat Harbour Drive										
Clint Street	SNBOATE02	4.10	4.33	230	4.23	130	4.48	380	4.39	290
Owen Cresc.	SNOWEN03	5.12	5.29	170	5.27	150	5.42	300	5.41	290
Within Retention Basin/ Lagoon										
D/S Smith St	SNLOWSSC01	2.25	3.71	N/A	3.22	N/A	4.17	N/A	3.64	N/A
D/S Riley St	SNLOWSSW01	2.70	3.71	N/A	3.22	N/A	4.17	N/A	3.64	N/A
Middle of area	SNLOWSS01	2.70	3.71	N/A	3.22	N/A	4.17	N/A	3.64	N/A
D/S Whittaker	SNROSS03	2.44	3.68	N/A	3.21	N/A	4.23	N/A	3.64	N/A

Note: *Refer Figure 20 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded
N/A shown for depths in areas where compliance with QUDM is not necessary

TABLE 4.20
Lagoon Levels
Addition of Elizabeth Street Lagoon

Location	Node	2 Year Event			100 Year Event		
		Ultimate Level (mAHD)	With Basin (mAHD)	Diff. (mm)	Ultimate Level (mAHD)	With Basin (mAHD)	Diff. (mm)
Northern Lagoons							
Lagoon 73	SNLOWN01	1.84	1.81	-30	2.90	2.88	-20
Lagoon 60	SNLOWN06	1.98	1.93	-50	2.90	2.88	-20
Central Lagoon							
Lagoon 50	SNLOW01	2.15	2.09	-60	3.02	3.01	-10
Southern and Eastern Lagoons							
Lagoon 40	SNLOWS11	2.18	2.12	-60	3.02	3.01	-10
Lagoon 30	SNLOWS09	2.19	2.12	-70	3.03	3.01	-20
Lagoon 20-Botanic Gardens	SNLOWS06	2.39	2.35	-40	3.03	3.01	-20
Kondari Resort Lagoon 21	SNLOWS01	1.62	1.61	-10	3.03	3.01	-20

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

4.5.2 Hammond Street and Lavell Street

As shown on **Figure 3- 100 Year Event Inundation Plan**, excessive flooding occurs at the low point in Hammond Street between Park Street and Bollero Street and at the intersection of Lavell Street and Boat Harbour Drive. In the case of the Hammond Street low point, water travels overland through houses to Boat Harbour Drive.

Proposed relief drainage works for the area are shown on **Figure 21- Hammond Street and Lavell Street Relief Drainage Works**.

The reduction in flood level afforded by the proposed relief drainage works is summarised in **Table 4.21- Hammond and Lavell Street Relief Drainage Works**. Comprehensive results are presented in **Appendix C- Model Results** in the following tables:

Table C61 **Peak Water Levels, 2 and 100 Year Events, Surface System**
Table C62 **Peak Water Levels, 2 and 100 Year Events, Underground System**
Table C63 **Peak Flows and Velocities, 2 and 100 Year Events, Surface System**
Table C64 **Peak Flows and Velocities, 2 and 100 Year Events, Underground System**

TABLE 4.21
Hammond and Lavell Street Relief Drainage Works
Result Summary

Location	Model Node	Ground Level	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
		(m AHD)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Hammond St between Park St and Bellero St	SNHAMM05	4.69	5.02	330	4.94	250	5.22	530	5.18	490
Boat Harbour Drive bet. Edith St & Lavell St	SNBOAT04	4.31	4.46	150	4.44	130	4.52	210	4.51	200
Hammond St at Bellero St	SNBELL04	5.00	5.21	210	5.21	210	5.33	330	5.33	330
Boat Harbour Dr at Lavell St	SNLAVE02	4.30	4.48	180	4.41	110	4.62	320	4.54	240

Note: *Refer Figure 21 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded

In the case of Hammond Street, existing ground levels and downstream drainage works preclude the use of any pipe larger than 600 mm diameter. While the proposed works (adding two 600 mm diameter pipes to the existing two 600 mm diameter pipes) would provide some alleviation of nuisance flooding, it would not be practicable to provide sufficient drainage to prevent flooding of the properties downstream of the low point in Hammond Street. Only by acquiring the properties downstream of the low point would it be possible to obtain an unobstructed overland flow path to Boat Harbour Drive. The cost of this scenario has not been included in the costings.

The estimated cost to complete the works in Hammond Street and Lavell Street is $166,000.

4.5.3 Tavistock Street

The depth of flooding at the low point in Charlton Esplanade near Tavistock Street is excessive. The low point drains via the existing Tavistock Street outlet. It was found that upgrading the drainage between the low point and Tavistock Street would provide an adequate reduction in peak flood level at the low point (refer **Figure 22- Tavistock Street Augmentation**).

The reduction in flood level afforded by the proposed relief drainage works is summarised in **Table 4.22- Tavistock Street Relief Drainage Works**. More comprehensive results are presented in **Appendix C- Model Results** in the following tables:

Table C65 **Peak Water Levels, 2 and 100 Year Events, Surface System**
Table C66 **Peak Water Levels, 2 and 100 Year Events, Underground System**
Table C67 **Peak Flows and Velocities, 2 and 100 Year Events, Surface System**
Table C68 **Peak Flows and Velocities, 2 and 100 Year Events, Underground System**

TABLE 4.22
Tavistock Street Relief Drainage Works
Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
			Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Low point in Charlton Esp. west of Tavistock St	SNTADE03	3.76	3.96	200	3.86	100	4.29	530	3.96	200
Charlton Esp further to the west	SNTADE04	4.17	4.23	60	4.23	60	4.29	120	4.26	90
Charlton Esp. at Tavistock St.	SNTAVN02	4.28	4.29	10	4.29	10	4.30	20	4.30	20

Note: Refer Figure 22 for pipe dimensions and locations.
Locations where depth of water is greater than desirable shaded

The estimated cost to complete the works is $45,300.

4.5.4 Torquay Road and Denmans Camp Road

The calculated 100 year flood level in the field inlet located upstream of Denmans Camp Road is sufficiently high to produce overland flow and flooding of the property immediately downstream.

To overcome this, pipes could be added to the existing pipes at Denmans Camp Road and Torquay Road (refer **Figure 23- Torquay Road/ Denmans Camp Road Relief Drainage Works**).

The reduction in flood level afforded by the proposed relief drainage works is summarised in **Table 4.23- Torquay Road/ Denmans Camp Road Relief Drainage Works**. Comprehensive results are presented in **Appendix C- Model Results** in the following tables:

Table C69 **Peak Water Levels, 2 and 100 Year Events, Surface System**
Table C70 **Peak Water Levels, 2 and 100 Year Events, Underground System**
Table C71 **Peak Flows and Velocities, 2 and 100 Year Events, Surface System**
Table C72 **Peak Flows and Velocities, 2 and 100 Year Events, Underground System**

TABLE 4.23
Torquay Road/ Denmans Camp Road Relief Drainage Works Result Summary

Location	Model Node	Adjacent Ground Level (m AHD)	Invert Level (m AHD)	Maximum Water Level 2 Year Event (m AHD)		Maximum Water Level 100 Year Event (m AHD)	
				Ultimate Case	With Relief Works	Ultimate Case	With Relief Works
Field Inlet east of Denmans Camp Road	SNLOWC07	4.7	2.90	4.36	4.05	4.94	4.72
Downstream of Denmans Camp Road	SNLOWC08	4.5	2.55	3.80	3.73	4.28	4.16
Upstream of Torquay Road	SNLOWC10	4.6	2.46	3.71	3.54	4.26	4.11
Downstream of Torquay Road	SNLOWC11	4.6	2.28	3.16	3.19	3.40	3.45

Note: *Refer Figure 23 for pipe dimensions and locations.*
Locations where depth of water is greater than desirable shaded
Invert level refers to pipe system which drains area, Ground level refers to actual surface level in vicinity of location.

It can be noted that a small bund would be required downstream of the field inlet to prevent overland flow occurring for the 100 year event.

The estimated cost to complete the works is $135,700. It is recognized that the completion of the works would be difficult due to the presence of existing buildings and would be of lower priority than other areas given the adequate performance of the system for the 2 year event.

4.5.5 Caltex Site

It has been proposed to drain a 5.7 hectare area (the Caltex site) of the adjacent eastern catchment via the Kondari resort lagoon (refer **Figure 4- Relief Drainage Works Key Plan**).

The addition of the Caltex site to the Lowlands catchment was modelled and the impact on lagoon levels determined. Comprehensive results are presented in **Appendix C- Model Results** in **Table C73- Peak Water Levels, 2 and 100 Year Events, Surface System.**

The drainage of the Caltex site via the Kondari Resort Lagoon was found to produce the following flood level increases compared to the ultimate basecase:

- 2 Year Event — Kondari Lagoon — 9 mm increase
  Other Lagoons — 0 mm increase
- 100 Year Event — All Lagoons — 8 mm increase.

Given this result, it was concluded that the drainage of the Caltex site would only have a minimal impact on levels in the Lowlands Lagoons and that the mitigation measures proposed for the lagoons would compensate for the increase in flood level.

4.6 Relief Drainage Works to be Designed

Excessive flood depths were noted at a number of other locations. However, it was considered that standard methods of analysis could be employed to derive drainage solutions for the areas and therefore were outside the scope of the study. These areas were:

- Torquay Road between Down Street and Queens Road (node SNQUDOS02)
- Mackean Road between Down Street and Queens Road (node SNQUEDO01)
- East Street and Torquay Road east of Queens Road (node SNBOOQU04 and SNDEQU02)
- Torquay Terrace near Fraser Street (node SNLINCO02)

The Moonbi Street to Parkway Drive system was identified as flooding during the 100 year event. As the area was recently the subject of relief drainage works, additional works were not considered.

4.7 Adopted Relief Drainage Works

4.7.1 Modelled Works

The performance of the relief drainage works considered for the catchment was reviewed and the most promising options identified. These options were reviewed by Council, allowing a set of relief drainage works most likely to be implemented to be specified. In order to allow the flood level reductions afforded by the combination of relief drainage works to be quantified, the works were modelled. The combination of relief drainage works adopted for the purposes of the study were as follows:

- Bideford Street augmentation;
- Frank Street augmentation;
- Optimisation of flood storage capacity of lagoons;
- Excavation of lagoon to the west of Robert Street;
- Construction of a bund at Robert Street;
- Upgrade of Macks Road and Robert Street drainage;
- Ann Street drainage upgrade; and
- Churchill Street augmentation.

The location of the works is shown on **Figure 24- Adopted Relief Drainage Works, Key Plan** and **Figure 25- Adopted Relief Drainage Works, Macks Road to Alexander Street**.

The relief drainage works modelled for the combined analysis were modified from those considered initially (refer Sections 4.2 to 4.6) to reflect the detailed design of certain works and the likelihood that some mitigation options may not be able to be fully implemented. The modifications made to each of the relief drainage measures considered in the analysis are detailed below.

- ***Bideford Street augmentation***

  The modelling of the augmentation of Bideford Street was adjusted to reflect the detailed design of the Bideford Street works, as presented in Hervey Bay City Council drawing series 337-11. A 3 x 1.2 metre box culvert was adopted for Bideford Street instead of the three 1.5 x 1.2 metre box culverts initially modelled. As the invert level of the box culvert is relatively high, the design includes a 750 mm diameter low flow pipe extending from Charlton Esplanade. This pipe was included in the model.

- ***Frank Street augmentation***

  The modelling of the augmentation of Frank Street was adjusted to reflect the detailed design of the Frank Street works, as presented in Cardno MBK drawing series 2919/43-01. As per the initial modelling, three 2.4 x 0.9 metre box culverts will be constructed.

- ***Optimisation of flood storage capacity of lagoons***

  A study was completed for Council in 1997 that identified potential works to maximise the flood storage capacity of the lagoons (Council drawings 2001-095 C1 to C9). Due to the development that has occurred on the banks of the lagoons, it is recognised that it will not be possible to complete some of the works previously identified. These restrictions to the extent of works possible are in addition to the restrictions considered in Section 4.2.

  In particular, it is likely that it will not be possible to construct revetment walls adjacent to Lagoons 30, 40, 60, and 73 (refer to **Figure 10- Lagoon Improvement Overview** for the location of lagoons).

  Further, development has occurred on the property to the east of Alexander Street and has involved modification to the existing lagoon. The flood storage capacity of the new lagoon configuration was assessed from as-constructed survey of the lagoon supplied by Council.

  The revised lagoon storage volumes, which should be readily achievable, are presented in **Table 3.3- Lagoon Volumes**.

- ***Excavation of lagoon to the west of Robert Street***

  Council has indicated that it is likely that it will construct a lagoon in the area of land to the west of Robert Street. The lagoon, as shown on **Figure 5- Robert Street Bund and Lake**, was modelled as per the initial analysis (refer Section 4.2).

- ***Construction of a bund at Robert Street***

  A bund is likely to be constructed at Robert Street to reduce the volume of runoff entering the lagoons. In order to allow a connection to the lagoon to be formed to the west of the bund (refer above), a pipe will be provided beneath the bund, as shown on **Figure 5- Robert Street Bund and Lake**.

For the analysis, a 600 mm diameter pipe with an invert level of RL 0.5 m AHD was assumed. Although such a diameter is not ideal from a fish passage perspective (refer Section 4.8.2), in this case it is necessary to use a relatively small diameter pipe in order to limit the volume of runoff entering the lagoons from the west.

- ***Upgrade of Macks Road and Robert Street drainage***

  Of the available options for Macks Road and Robert Street, the option involving the drainage of both areas to the lagoons is considered to be the most attractive. This was previously modelled as Option C (refer Section 4.4.3 and **Figure 15-Robert Street Relief Drainage Works, Option C- Drain Robert Street and Macks Road to Lagoons**).

  Council has completed construction of Stage 1 of the relief drainage works proposed for Robert Street. The model was adjusted to reflect the detailed design for the works completed and proposed for Robert Street, as detailed in Council drawing series 2001-183. The design for Macks Road was assumed to be the same as that modelled initially (refer Section 4.4.3 and **Figure 25-Adopted Relief Drainage Works, Macks Road to Alexander Street**).

  It can be noted that the Robert Street works include the construction of new swales. Care should be taken when interpreting the water depths presented for the area as they relate to the invert of the swale rather than the invert of the road. The roads affected are:

  - Robert Street between Cypress Street and Truro Street (links slrob03, slrob04, slrob05);
  - Eric Street between Cypress Street and Truro Street (links sleric01, sleric02, and sleric03);
  - Ocean Street between Robert Street and Eric Street (link slerob02);
  - View Street between Macks Road and Robert Street (link slroma02); and
  - Ocean Street between Macks Road and Robert Street (link slroma03).

- ***Ann Street drainage upgrade***

  For the area between Eric Street and Alexander Street (referred to as the Ann Street drainage works), consideration was given to draining the catchment to either the ocean or the lagoon system.

  For both scenarios, it was recognised that the pipe sizes necessary to obtain depths of flooding that complied with the requirements of the Queensland Urban Drainage Manual for major flood events would be prohibitively expensive. Alternate solutions that involved obtaining acceptable flood depths wherever possible for the minor event and reasonable reductions in flood level for the major event were therefore defined. The latter minor event solutions were considered to be preferable given the significant reduction in construction cost associated with the minor event solutions and the relatively small additional flood level reduction obtained from the major event solution.

Following a review of the available drainage solutions, Council indicated that it would most likely implement a combination of the drainage solutions, as shown on **Figure 25- Adopted Relief Drainage Works, Macks Road to Alexander Street**:

- *Option A1 (drain to ocean, minor event solution)*

  Option A1 included the retention of the existing system in Ann Street and the reduction of the catchment area draining to the system. New pipe work would be constructed in Cypress Street to connect the existing gully pits at the intersection of Witt Street and Cypress Street to the Ann Street system. The remainder of the Option A1 works would not be constructed.

- *Option B1 (drain to lagoons, minor event solution)*

  Option B1 includes the construction of pipe systems in Ann Street and Crown Street/ Brown Street/ Alexander Street. Of these systems, the works proposed in Ann Street between Charlton Esplanade and Cunningham Street would not be completed due to the adoption of the Option A1 solution in this area. Further, the pipe proposed on Charlton Esplanade to the east of Crown Street would not be constructed and the existing outfall between Crown Street and Alexander Street would be retained.

During the review of available relief drainage measures, it was noted that the depth of ponded water in Pebble Court largely governs the size of the drainage required in Ann Street. In order to allow the size of pipe constructed in Ann Street to be minimised, Pebble Court could be drained via a separate drainage system. The separate drainage system would connect Pebble Court with the existing drainage system in Truro Court via the caravan park to the west of Pebble Court. Due to the limited depth of cover available for a pipe between Pebble Court and Truro Street, a box culvert solution was considered for the analysis.

Further, the concept design provided by Council for the drainage of Pebble Court included the retention of the existing 375 mm diameter pipe linking Keys Avenue and Ann Street. Modelling determined that it would be more cost effective to upgrade this pipe than adopt a pipe size in Ann Street larger than would otherwise be necessary.

- ***Churchill Street augmentation***

  As shown on **Figure 7- Churchill Street Augmentation**, it is proposed to duplicate the existing box culvert that discharges water from the Kondari Resort lagoon (Lagoon 21) to the ocean. No changes were made to the design initially modelled (refer Section 4.2) for the combined analysis.

- ***Caltex site***

  The drainage solution adopted for a site (known colloquially as the Caltex site) at the upstream end of the adjacent Urangan catchment is to drain runoff from the site to the lagoon system (refer Section 4.5.5 and **Figure 24- Adopted Relief Drainage Works**). This allows the size of drainage infrastructure required for the adjacent catchment to be reduced.

The runoff hydrographs prepared for the catchment were revised to include the runoff from a total area of 5.26 hectares. Of this area, 4.43 hectares was assumed to drain directly to the Kondari Resort lagoon (node SNLOWS02), with the remaining 0.83 hectares draining to Dayman Street (node SNLOWSSE02).

It can be noted that the Ann Street works will result in an increased volume of runoff entering the lagoon system. In order to prevent an increase in flood level in the lagoons, it is necessary to complete the Churchill Street augmentation and the excavation of the connection channel through the Botanic Gardens (part of the works considered for the optimisation of lagoon areas) prior to the upgrading of the Ann Street drainage system.

4.7.2 Results of Modelling

The peak flood levels calculated within the lagoons and at key points in Tooan Tooan Creek for the combination of works identified in Section 4.7.2 are presented in **Table 4.24- Lagoon Levels, Combined Relief Drainage Works.**

Peak flood levels calculated for the region between Macks Road and Alexander Street are presented in **Table 4.25- Macks Road to Alexander Street, Combined Relief Drainage Works Result Summary**.

Comprehensive results for the analysis are presented in **Appendix C- Model Results**:

Table C1 **Peak Water Levels, 2 and 100 Year Events, Surface System**
Table C2 **Peak Water Levels, 2 and 100 Year Events, Underground System**
Table C3 **Peak Flows and Velocities, 2 and 100 Year Events, Surface System**
Table C4 **Peak Flows and Velocities, 2 and 100 Year Events**

TABLE 4.24
Lagoon Levels
Combined Relief Drainage Works

Location	Node	2 Year Event				100 Year Event			
		Existing (mAHD)	Ultimate (Prior to Works) (mAHD)	Level After Works (mAHD)	Diff. (mm)	Existing (mAHD)	Ultimate (Prior to Works) (mAHD)	Level After Works (mAHD)	Diff. (mm)
Northern Lagoons									
Lagoon 73	SNLOWN01	1.71	1.84	1.83	-10	2.85	2.90	2.77	-130
Lagoon 60	SNLOWN06	2.19	1.98	1.95	-30	2.86	2.90	2.77	-130
Central Lagoons									
Lagoon 50	SNLOW01	2.23	2.15	2.09	-60	2.98	3.02	2.82	-200
Southern and Eastern Lagoons									
Lagoon 40	SNLOWS11	2.24	2.18	2.10	-80	2.99	3.02	2.83	-190
Lagoon 30	SNLOWS09	2.24	2.19	2.11	-80	2.99	3.03	2.83	-200
Lagoon 20- Botanic Gardens	SNLOWS06	2.38	2.39	2.03	-360	2.99	3.03	2.82	-210
Kondari Resort Lagoon 21	SNLOWS01	1.81	1.62	1.67	50	2.98	3.03	2.82	-210
Tooan Tooan Creek to West of Lagoons									
To west of Robert Street	SNLOW03	2.42	2.44	2.10	-340	2.99	3.02	2.91	-110
Bideford Street	SNLOW14	2.68	2.73	2.31	-420	3.10	3.16	2.95	-210
Denmans Camp Road	SNLOW28	2.73	2.80	2.64	-160	3.20	3.24	2.99	-250
Frank Street	SNLOW33	2.71	2.81	2.55	-260	3.21	3.25	2.96	-290

Note: *Refer Figure 10 for location of lagoons*
Diff. refers to change in level compared to ultimate level. Positive values indicate an increase in flood level while negative values indicate a reduction in flood level.

TABLE 4.25
Macks Road to Alexander Street
Combined Relief Drainage Works Result Summary

Location	Model Node	Ground Level (m AHD)	2 Year Event				100 Year Event			
			Ultimate Case		With Relief Works		Ultimate Case		With Relief Works	
			Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)	Max Water Level (m AHD)	Max Water Depth (mm)
Locations on Robert Street (refer notes)										
Charlton Esp.	SNROB08	4.24	4.27	30	4.26	20	4.30	60	4.29	50
Cypress Street	SNROB06	3.39	3.48	90	3.41	80	3.62	230	3.47	130
Ocean Street	SNROB05	3.08	3.44	360	3.07	70	3.62	540	3.47	470 (320)
View Street	SNROB04	3.21	3.44	230	3.30	50	3.61	400	3.46	210 (<0)
Truro Street	SNROB03	3.47	3.49	20	3.40	20	3.59	120	3.43	50
Other Locations										
Eric St between Charlton Esp & Cypress St	SNERIC05	3.24	3.38	140	3.38	140	3.66	420	3.66	420
Cnr Eric St and Truro St	SNERIC01	3.20	3.36	160	3.28	80	3.46	260	3.39	190
Locations on Macks Road										
Charlton Esp.	SNMACKS05	3.61	3.70	90	3.68	70	3.77	160	3.73	120
Cypress Street	SNMACKS04	3.22	3.46	240	3.32	100	3.63	410	3.55	330
Ocean Street	SNMACKS03	3.27	3.44	170	3.32	50	3.62	350	3.55	280
View Street	SNMACKS02	3.38	3.43	50	3.40	20	3.57	190	3.49	110
Truro Street	SNMACKS01	3.22	3.29	70	3.28	60	3.40	180	3.32	100
Locations On Ann Street										
Charlton Esp	SNANNN09	3.36	3.47	110	3.45	90	3.69	330	3.66	300
Sag between Charlton Esp & Cypress St	SNANNN08	3.35	3.47	120	3.47	120	3.69	340	3.66	310
Cypress Street	SNANNN07	3.91	3.94	30	3.94	30	3.97	60	3.97	60
Cunningham St	SNANNN06	3.25	3.53	280	3.33	80	3.82	570	3.55	300
Keys Avenue	SNANNN04	3.45	3.54	90	3.48	30	3.82	370	3.56	110
Truro Street	SNANNN03	3.49	3.60	110	3.57	80	3.79	300	3.62	130
Locations off Ann Street										
Cypress St at Witt Street	SNWIL01	3.34	3.54	200	3.46	120	3.70	360	3.66	320
Low point in Cunningham St	SNBRANN02	3.15	3.53	380	3.27	120	3.82	670	3.55	400
Keys Av near Ann Street	SNKEYS03	3.28	3.54	260	3.38	100	3.82	540	3.56	280
Keys Av at Pebble Court	SNKEYS01	3.29	3.63	340	3.35	60	3.83	540	3.63	340
Pebble Court	SNDEB01	3.13	3.63	500	3.34	210	3.83	700	3.63	500
Anembo Drive near Ann St	SNALAN04	2.75	2.87	120	2.87	120	3.12	360	2.98	230
Anembo Drive at Rosalind Court	SNALAN03	3.21	3.25	40	3.26	50	3.30	90	3.29	80
Locations On Crown and Brown Streets										
Charlton Esp.	SNCROWN01	3.67	3.73	60	3.73	60	3.80	130	3.76	90
Cypress Street at Crown Street	SNCROWN02	3.29	3.48	190	3.41	120	3.69	400	3.66	370
Cypress Street at Brown Street	SNBROWN03	3.60	3.66	60	3.65	50	3.82	220	3.70	100
Cunningham St	SNBROWN02	3.31	3.53	220	3.39	80	3.82	510	3.55	240
Christine Ave	SNBROWN01	3.46	3.55	90	3.53	70	3.82	360	3.57	110
Locations off Crown and Brown Streets										
Charlton Esp bet. Crown St & Alexander Sr	SNALCR01	3.19	3.40	210	3.39	200	3.69	500	3.66	470
Alexander St at Christine Av	SNALEXN04	2.97	3.23	260	3.06	90	3.62	650	3.22	250
Low point in Truro St	SNTRURO01	3.28	3.48	200	3.42	140	3.87	590	3.79	510

Note: Refer Figure 25 for pipe dimensions and locations.
Locations where depth of water is greater than desirable shaded
Surface links on Robert Street include designed swales. Quoted depths for Robert Street relate to depth in swale rather than depth on road pavement. Figures in brackets indicate depth above minimum roadway elevation.

The results presented in Table 4.24 and Table 4.25 were compared to those obtained from the initial analysis (refer Sections 4.2 to 4.6). In general, the results obtained from the combination of relief drainage works were consistent with the results calculated for each of the relief drainage options.

The flood level reduction achieved in the lagoons was found to be slightly less than that calculated previously (refer Tables 4.16 and 4.17). For the 2 year event, the reduction in flood level due to relief drainage works was found to decrease by about 40 to 90 mm. For the 100 year event, the impact on the flood level reduction afforded by the relief works was found to be nominal (10 to 20 mm), with the peak flood level reached in the lagoons less than RL 2.9 m AHD. This was attributed to the reduction in available lagoon area and the fact that the Churchill Street outfall is more effective at draining the lagoon system during major flood events when flow can freely travel from the main lagoons through the Botanic Gardens to the Kondari Resort Lagoon.

In Tooan Tooan Creek to the west of Robert Street, calculated flood levels for the 2 and 100 year events were found to be slightly higher than those calculated previously (refer Tables 4.5 and 4.6). Peak flood levels for the 100 year event were found to be between 20 and 40 mm higher than those calculated previously, resulting in a peak flood level between RL 2.9 m AHD and RL 3 m AHD. This was attributed to the fact that the size of the drainage to be constructed at Bideford Street is less than modelled previously. However, the resultant flood levels were considered to be acceptable and it was concluded that an increase in the size of works at Bideford Street could not be justified.

The results obtained in the area between Macks Road and Alexander Street were found to be generally consistent with the flood levels calculated previously (refer Section 4.4 and Tables 4.11 and 4.14). As the works at Robert Street do not include a pipe to drain the low point in Eric Street between Charlton Esplanade and Cypress Street, the depth of flooding at this location for the 100 year event is relatively high (420 mm). However, as the existing depth of flooding for the 2 year event is within acceptable limits, it could be argued that the depth of flooding for the 100 year event is acceptable given the relatively high cost associated with draining the low point via Robert Street.

A relatively high depth of flooding was also calculated for the 100 year event at the intersection of Robert Street and Ocean Street. However, as the constructed works were found to produce a significant reduction in flood level for the 2 year event (greater than that initially proposed), the depth of flooding calculated for the 100 year event was considered to be acceptable.

The extent of inundation calculated for the combined relief drainage works is presented in **Figure 2- 2 Year Flood Map** and **Figure 3- 100 Year Flood Map**. With reference to the inundation plan, it can be noted that properties shown as being inundated, particularly in the region between Macks Road and Alexander Street, can be filled without affecting the calculated flood levels. Modelling assumed that individual lots would be filled at some point in the future as the lots are redeveloped.

The estimated cost to complete the combination of relief drainage works (excluding lagoon enlargement) is $6,956,000.

4.8 Road Crossings

4.8.1 Flood Immunity

Table 4.26- Flood Immunity of Road Crossings presents a summary of the flood levels predicted at each of the road crossings of Tooan Tooan Creek and the Lowlands Lagoons.

The table indicates that the completion of relief drainage works will provide immunity to flooding for the 100 year event at a number of crossings. Following the completion of the combination of works identified in Section 4.7 (refer **Figure 24- Adopted Relief Drainage Works**), the crossings that would be flooded by the 100 year event would be reduced to:

- Denmans Camp Road (130 mm depth of flooding for the 100 year event);
- Fraser Street (30 mm depth of flooding for the 100 year event);
- Ann Street northern crossing (270 mm depth of flooding for the 100 year event); and
- Alexander Street (850 mm depth of flooding for the 100 year event).

It can be noted that the 1994 Austroads publication *Waterway Design, A Guideline to the Hydraulic Design of Bridges, Culverts and Floodways* defines road closure as occurring when the depth of flooding across a road exceeds 300 mm. Flooding to a lesser depth is not generally considered to preclude the use of a road by emergency vehicles.

Based on this definition, only the Alexander Street crossing would be closed during the 100 year event. Ann Street would experience a significant depth of flooding but should still be trafficable by emergency vehicles. However, in both cases alternate routes are available to act as escape routes and to access properties within the catchment.

It can also be noted that the report does not recommend the upgrading of any existing crossing due to the relatively low head loss associated with the majority of the crossings.

TABLE 4.26
Flood Immunity of Road Crossings

Road	Node	Minimum Road Level (m AHD)	Peak Water Levels (Ultimate Base Case)		Comments
			2 Year Event (mAHD)	100 Year Event (mAHD)	
Charlton Esplanade	SNLOW49	3.17	1.97	2.63	Immune to flooding for the 100 year event
Zephyr Street	SNLOW45	2.94	2.05	2.70	Immune to flooding for the 100 year event
Queens Road	SNLOW38	3.16	2.34	2.80	Immune to flooding for the 100 year event
Frank Street	SNLOW33	3.14	2.81	3.25	Completion of relief drainage works (primarily at Bideford Street and Frank Street) will provide a 100 year flood level of about RL 2.96 m AHD and immunity to flooding for the 100 year event
Denmans Camp Road	SNLOW28	2.86	2.80	3.24	Completion of relief drainage works (primarily at Bideford Street and Frank Street) will provide a 100 year flood level of about RL 2.99 m AHD. The depth of flooding for the 100 year event would be reduced to 130 mm
Tavistock Street	SNLOW19	3.15	2.74	3.19	Completion of relief drainage works (primarily at Bideford Street and Frank Street) will provide a 100 year flood level of about RL 2.96 m AHD and immunity to flooding for the 100 year event
Bideford Street	SNLOW14	3.29	2.73	3.16	Immune to flooding for the 100 year event
Fraser Street	SNLOW09	2.89	2.71	3.13	Completion of relief drainage works (primarily at Bideford Street and Frank Street) will provide a 100 year flood level of about RL 2.92 m AHD. The depth of flooding for the 100 year event would be reduced to 30 mm.
Ann Street (Northern Crossing)	SNLOWN07	2.55	2.15	3.02	Completion of relief drainage works (primarily a bund at Robert St, enlarged lagoons and duplication of Churchill St drainage) will provide a 100 year flood level of RL 2.82 m AHD. Depth of flooding for the 100 year event would be reduced to 270 mm.
Ann Street (Southern Crossing)	SNLOWS14	2.90	2.16	3.02	Completion of relief drainage works (primarily a bund at Robert St, enlarged lagoons and duplication of Churchill St drainage) will provide a 100 year flood level of RL 2.82 m AHD and immunity to flooding.
Alexander Street	SNLOWN04	1.92	1.94	2.90	Completion of relief drainage works (bund at Robert St, Enlarged Lagoons and duplication of Churchill St) will provide 100 year flood level of RL 2.77 m AHD. Depth of flooding for the 100 year event would be reduced to 850 mm.

Note: Roads subject to flooding during the 100 year event following the completion of relief drainage works are shown shaded.
*Flood levels quoted for relief drainage works relate to the adopted set of works described in Section 4.7 and shown on **Figure 24- Adopted Relief Drainage Works**.*

4.8.2 Guidelines for the Design of Road Crossings

Although the primary focus of this investigation has been on the performance of the lagoons as flood mitigation devices, the lagoons also possess significant environmental and water quality values. If it is necessary to upgrade existing or construct new road crossings, due consideration needs to be given to environmental issues.

In particular, it is important that a continuous connection be established between the lagoons in order that circulation can occur to maintain water quality and allow fish to pass between the lagoons.

The recommended design criteria for road crossings are as follows (Fairfull and Witheridge 2003, pp 9-10, Cotterell 1998, p24):

- Minimum water depth of 0.2 to 0.25 metres, with a recommendation to locate the invert of at least one culvert (the so called "wet" cell) to match the bed of the lagoon. This implies a maximum invert level of RL 1.25 m AHD, although it would be preferable to set the invert of at least one culvert to match that of the bed of the lagoon.

- Location of elevated dry cells if possible to encourage terrestrial movement.

- Minimisation of changes to the channel's natural flow width and area (setting culvert inverts to maximise the geometric similarities to the natural channel profile from the bed of the culvert where possible).

- Natural bed material or rounded stone should be placed along the bed of wet cells to facilitate fish movement.

- Light penetration should be enhanced by the use of cells with maximised heights or diameters and possibly by the introduction of skylights or grated stormwater inlets for the wet cell.

- The order of preference for the type of crossing is as follows:

  - bridge
  - arch culvert
  - box culvert (bottomless if possible)
  - pipe culvert

  Box culverts are preferable to pipe culverts as they allow a constant channel width to be maintained regardless of the depth of flow (Cotterell 1998, p9). In the case of the lagoons, as the culverts will be permanently inundated and large diameter cells will be adopted for wet cells, this is considered to not be as significant an issue as it would be in other areas.

- Culvert slope should be as flat as possible and not exceed 1 in 100.

- Stream velocity should be a maximum of 1 m/s, preferably 0.3 m/s.

Summarising the above design criteria, a suitable design for a crossing of the Lowlands lagoons would comprise the following elements:

- Invert level of at least one cell to match the invert level of the lagoons (approximately RL 0.3 m AHD to RL 0.6 m AHD).
- Roughening of bed of wet cell by the placement of rocks or silt to mimic bed of lagoons.
- Adoption of large diameter pipe (1.5 metre diameter or larger) for the wet cell.
- Provision of an air gap of at least 0.3 metres above the standing water level in the lagoons, with consideration given to the inclusion of a skylight or grate to improve light penetration.

5.0 CONCLUSION

A detailed hydraulic model of the Lowlands catchment was created using the XP-UDD modelling package. The model was used to assess the impact upon flooding caused by ultimate catchment development and to consider a range of relief drainage options.

Relief drainage measures were considered for the following areas:

- The Lowlands Lagoons;
- Tooan Tooan Creek to the west of the lagoons;
- The flood prone area between Macks Road and Alexander Street (which could be drained either to the Ocean or to the lagoons); and
- Other drainage problem areas within the catchment.

The relief drainage options considered are summarized in **Table 5.1- Relief Drainage Summary**. The performance of the relief drainage works initially considered for the catchment was reviewed and the most promising options identified. These options were reviewed by Council, allowing a set of relief drainage works most likely to be implemented to be specified. The works are as follows:

- Bideford Street augmentation;
- Frank Street augmentation;
- Optimisation of flood storage capacity of lagoons;
- Excavation of lagoon to the west of Robert Street;
- Construction of a bund at Robert Street;
- Upgrade of Macks Road and Robert Street drainage;
- Ann Street drainage upgrade; and
- Churchill Street augmentation.

It can be noted that the Ann Street works will result in an increased volume of runoff entering the lagoon system. In order to prevent an increase in flood level in the lagoons, it is necessary to complete the Churchill Street augmentation and the excavation of the connection channel through the Botanic Gardens (part of the works considered for the optimisation of lagoon areas) prior to upgrading the Ann Street drainage system.

The relief drainage works modelled for the combined analysis were modified from those considered initially (refer Sections 4.2 to 4.6) to reflect the detailed design of certain of the works and the likelihood that some mitigation options may not be able to be fully realised (refer Section 4.7). The results presented in Table 5.1 relate to the modified set of relief drainage works.

The estimated cost to complete the combination of relief drainage works excluding lagoon works (refer Tables D19 to D22 in **Appendix D- Costing of Relief Works**) is $6,956,000.

TABLE 5.1
Relief Drainage Summary

Option		Reference Section in Main Report	Cost Excl GST (k$)	Advantages	Disadvantages
Lagoon Drainage Options					
Bund at Robert Street		4.2	Not Calc	Minimises runoff volume entering Lagoon system.	Directs runoff to western part of catchment.
Bund at Robert Street with pipe to connect lagoons to new lagoon west of Robert Street		4.7	8.4	Minimises runoff volume entering Lagoon system.	Directs runoff to western part of catchment.
Enlarged Lagoons		4.2 , 4.7	Not Calc	Maximises available flood storage and provides hydraulic connection between Lagoons and Kondari resort to maximize efficiency of Churchill Street outfall	Works possible have been constrained by development around boundary of lagoons
Extra Lagoon at Robert Street		4.2, 4.7	Not Calc	Can be used to provide additional storage volume in the event of works not being possible in the main lagoons	Relatively minor reduction in flood levels
Margaret St Augmentation		4.2	570	Would provide reduction in lagoon level without need for works at the existing outfall	Benefit compromised by high head loss at Ann Street. Length of drain longer than Churchill St
Churchill St Augmentation		4.2, 4.7	1,054	Provides significant flood level reduction, particularly in combination with other measures	Requires connection between southern lagoons and botanic gardens to obtain full benefit
Tooan Tooan Creek					
Bideford St Augmentation		4.3, 4.7	1,333	Provides good flood level reduction in vicinity of Bideford St and compensates for bund at Robert St	
Frank St Augmentation		4.3, 4.7	1,311	Provides good flood level reduction in vicinity of Frank St	
Macks Road to Alexander Street					
Macks Road	Option A- Drain to Ocean	4.4.2	430	Minimises runoff volume entering lagoons	Need to upgrade existing outfall. Not much cost saving compared to lagoon option
Robert Street	Option A- Drain to Ocean	4.4.3	1,077	Allows one outfall to be removed.	Need to upgrade existing outfall Greater flood depths for similar cost as Option B2
	Option B1- Drain to Lagoons Minor Event solution	4.4.3	750	Allows two outfalls to be removed	Relatively high flood depths for major event. Runoff directed to lagoons
	Option B2- Drain to Lagoons Major Event solution	4.4.3	925	Allows two outfalls to be removed. Provides lower flood depths than Option A at comparable cost	Runoff directed to lagoons
	Option C- Drain Macks Rd and Robert St to Lagoons	4.4.3	1,356	Allows three outfalls to be removed	Runoff directed to lagoons
	Drain Macks Rd and Robert St to Lagoons including detailed design of Robert St works	4.7	1,121	Allows two outfalls to be removed	Runoff directed to lagoons
Ann St	Option A1- Drain to Ocean, Minor Event Solution	4.4.4	2,003	Allows one outfall to be removed. Minimises runoff volume entering lagoons. Cost effective solution	Minor drainage solution Option B1 provides lower flood depths
	Option A2- Drain to Ocean, Major Event Solution	4.4.4	2,580	Allows one outfall to be removed. Minimises runoff volume entering lagoons.	Resultant flood depths still high
	Option B1- Drain to Lagoons, Minor Event Solution	4.4.4	2,567	Allows three outfalls to be removed. Cost effective solution	Runoff directed to lagoons
	Option B2- Drain to Lagoons, Major Event Solution	4.4.4	3,987	Allows three outfalls to be removed. Best reduction of flood depths	Runoff directed to lagoons. Cost prohibitive
	Combination of Option A1 and Option B1 with Pebble Court drained by separate system	4.7	2,129	Combination of best features of Options A1 and B1 to minimise construction cost while also draining majority of area to lagoons	Runoff directed to lagoons

Option	Reference Section in Main Report	Cost Excl GST (k$)	Advantages	Disadvantages
Other Drainage Works				
Elizabeth Street	4.5.1	1,278 + Land Acquisition	Provides reduction in main lagoon level. Minimises flooding in Elizabeth Street area	Cost prohibitive unless profit can be gained from fill sold from excavation
Hammond and Lavell Streets	4.5.2	166	Provides reduced incidence of flooding	Downstream flooding still unacceptable- would require land acquisition to solve
Tavistock Street	4.5.3	45	Reduces excessive depth of ponding	
Torquay Road/ Denmans Camp Road	4.5.4	136	Prevents overland through buildings for major event flooding	Difficult to install pipes beneath existing buildings.

Note: *Detailed costing information presented in Appendix D*
For location of relief drainage works, refer to Figure 4- Relief Drainage Works Key Plan and Figure 24- Adopted Relief Drainage Works
Recommended works shaded.

6.0 REFERENCES

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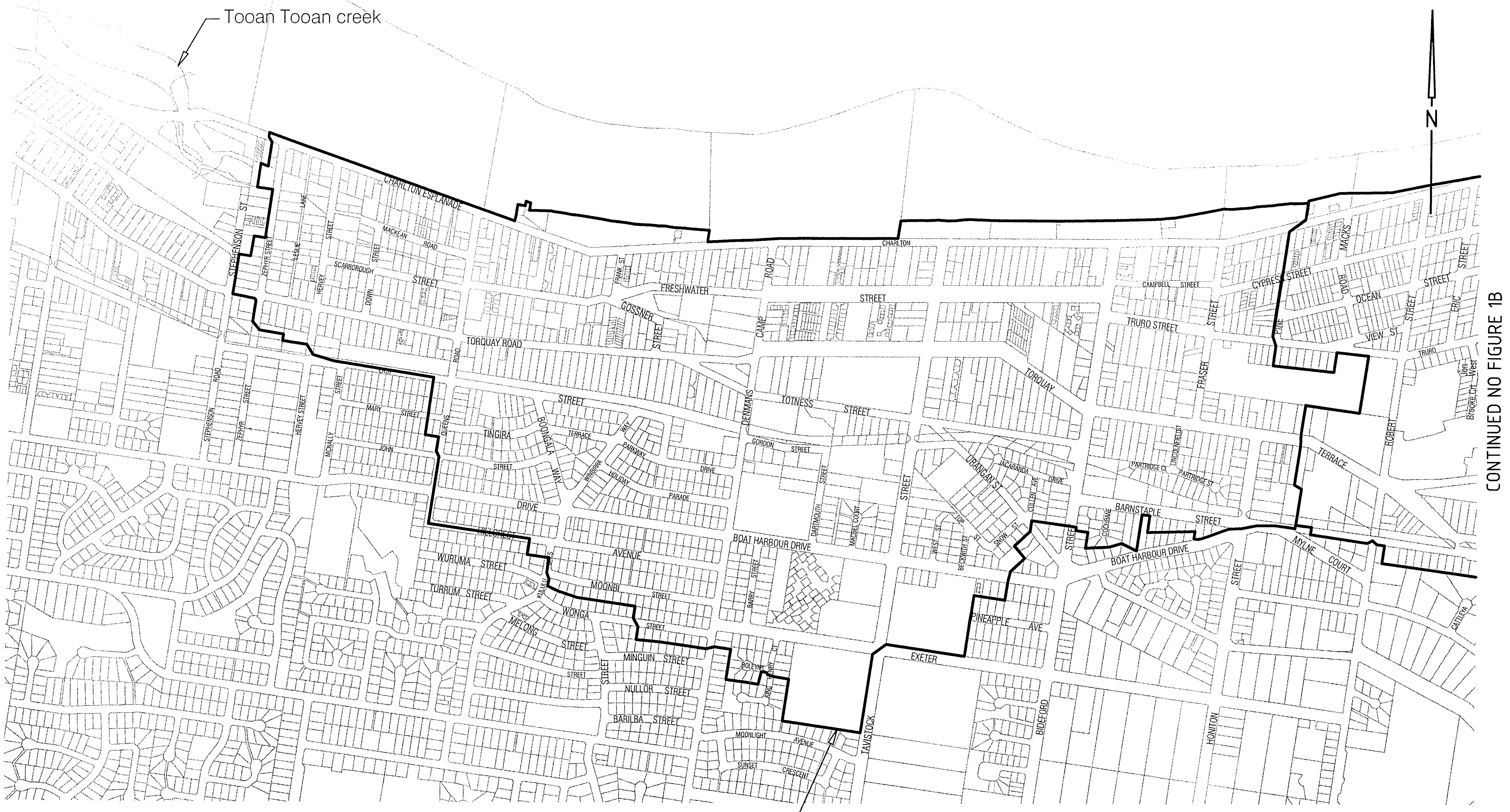
FIGURES



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Rev: Orig. Date: 21 November 2000

Project No.: 2919/20 (3)

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LOWLANDS LAGOONS DRAINAGE STUDY

STUDY AREA

FIGURE 1A

Cardno MBK

Rev: Orig. Date: 21 November 2000

Project No.: 2919/20 (3)

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LOWLANDS LAGOONS DRAINAGE STUDY

STUDY AREA

FIGURE 1B

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

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LOWLANDS LAGOONS

2 YEAR FLOOD MAP

FIGURE 2

LOWLANDS LAGOONS DRAINAGE STUDY
HERVEY BAY CITY COUNCIL

LOWLANDS LAGOONS

100 YEAR FLOOD MAP

FIGURE 3

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

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LOWLANDS LAGOONS DRAINAGE STUDY
HERVEY BAY CITY COUNCIL

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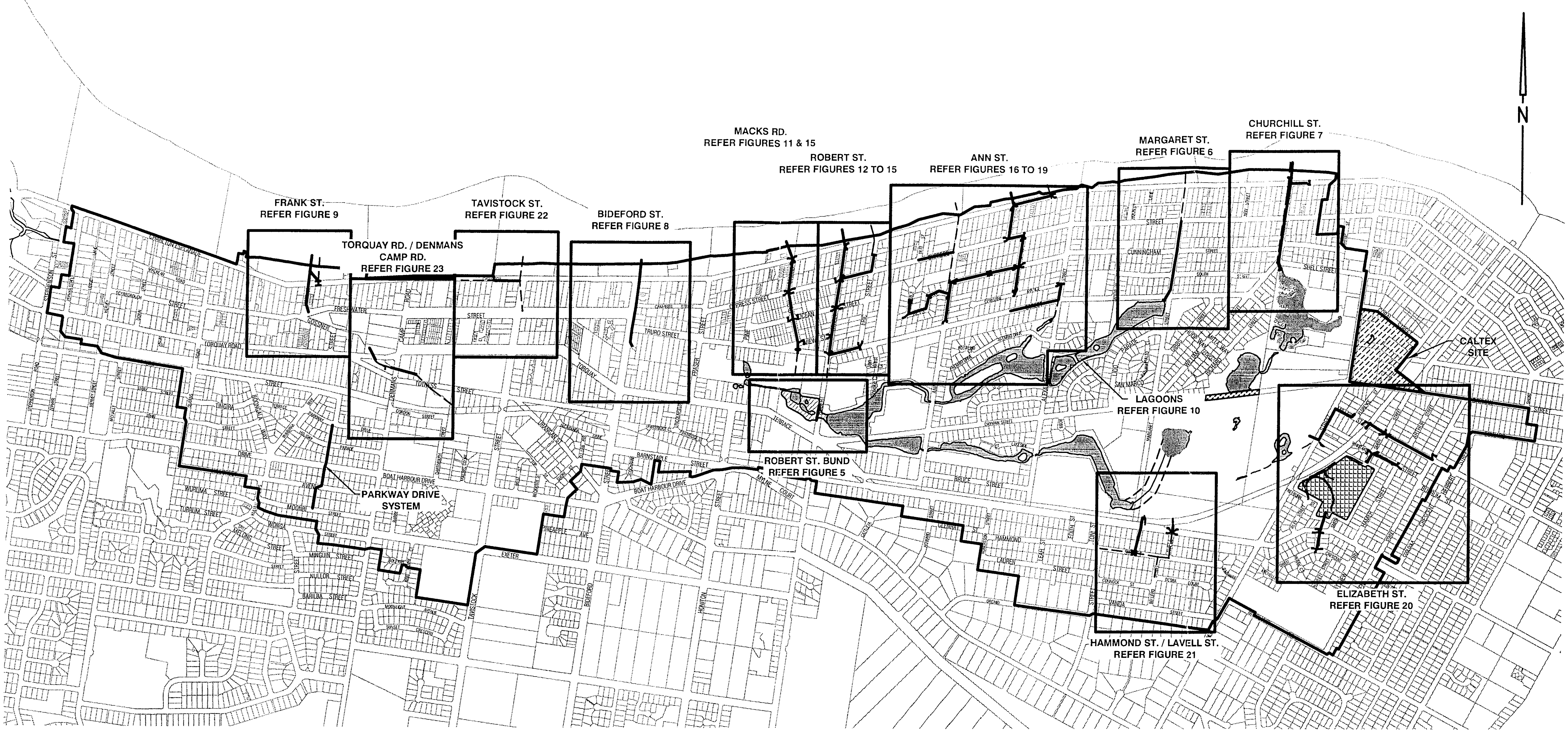

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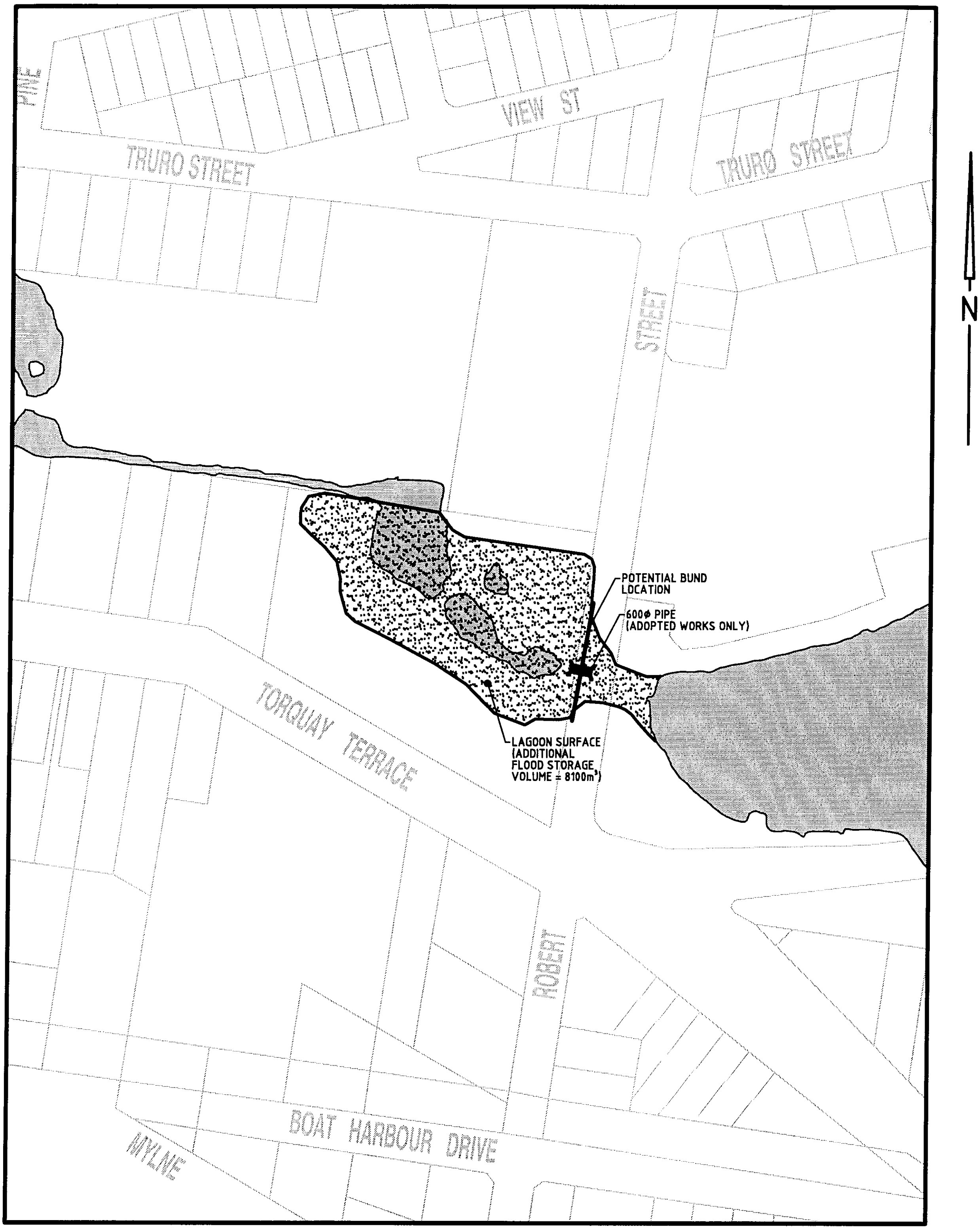
MODELLED RELIEF DRAINAGE SCENARIOS

KEY PLAN

FIGURE 4



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Rev: Orig. Date: 22 February 2002

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ROBERT STREET BUND & LAKE

FIGURE 5

Project No.: 2919/20

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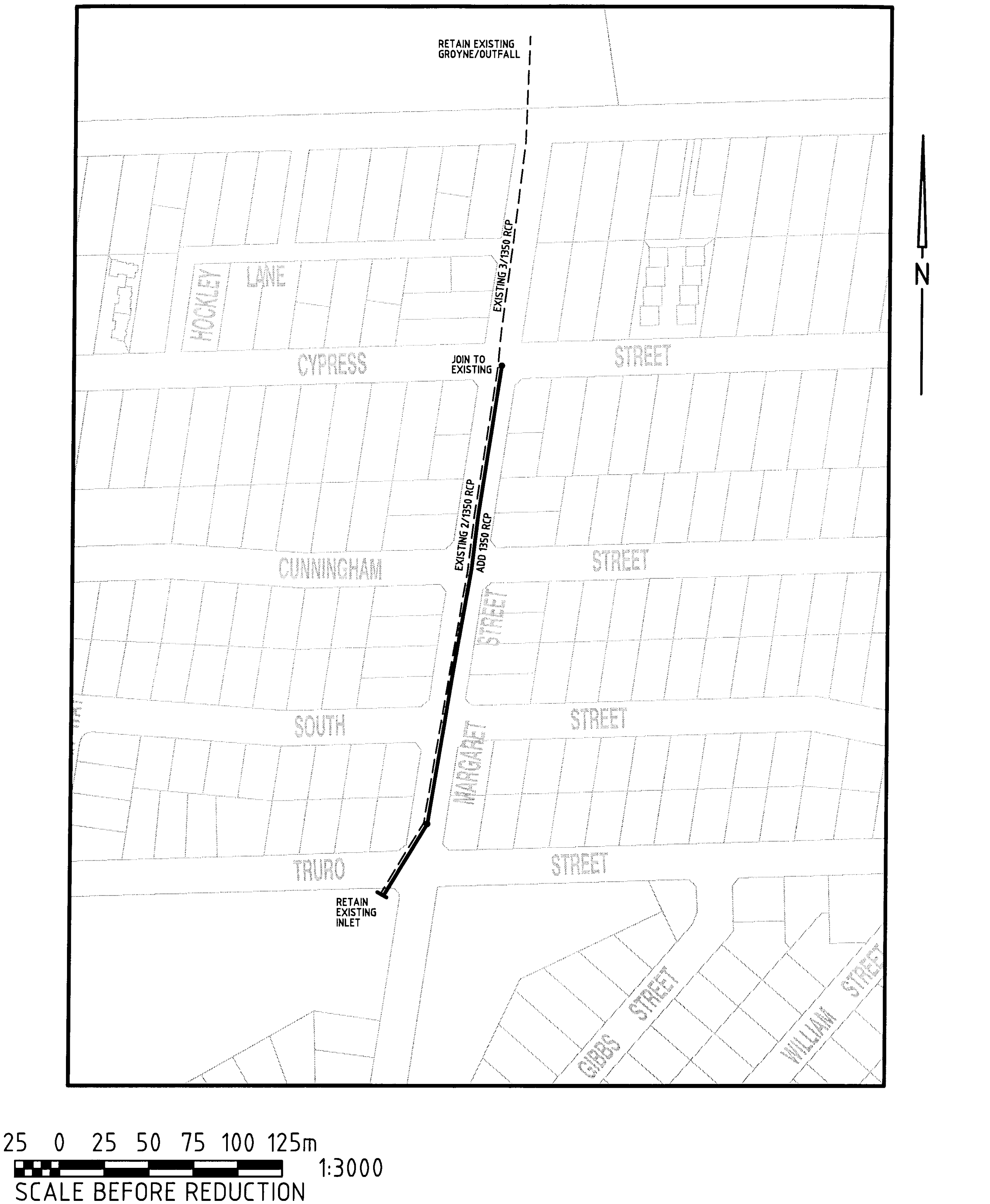

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Rev: Orig. Date: 22 February 2002

MARGARET ST. AUGMENTATION

FIGURE 6

Project No.: 2919/20

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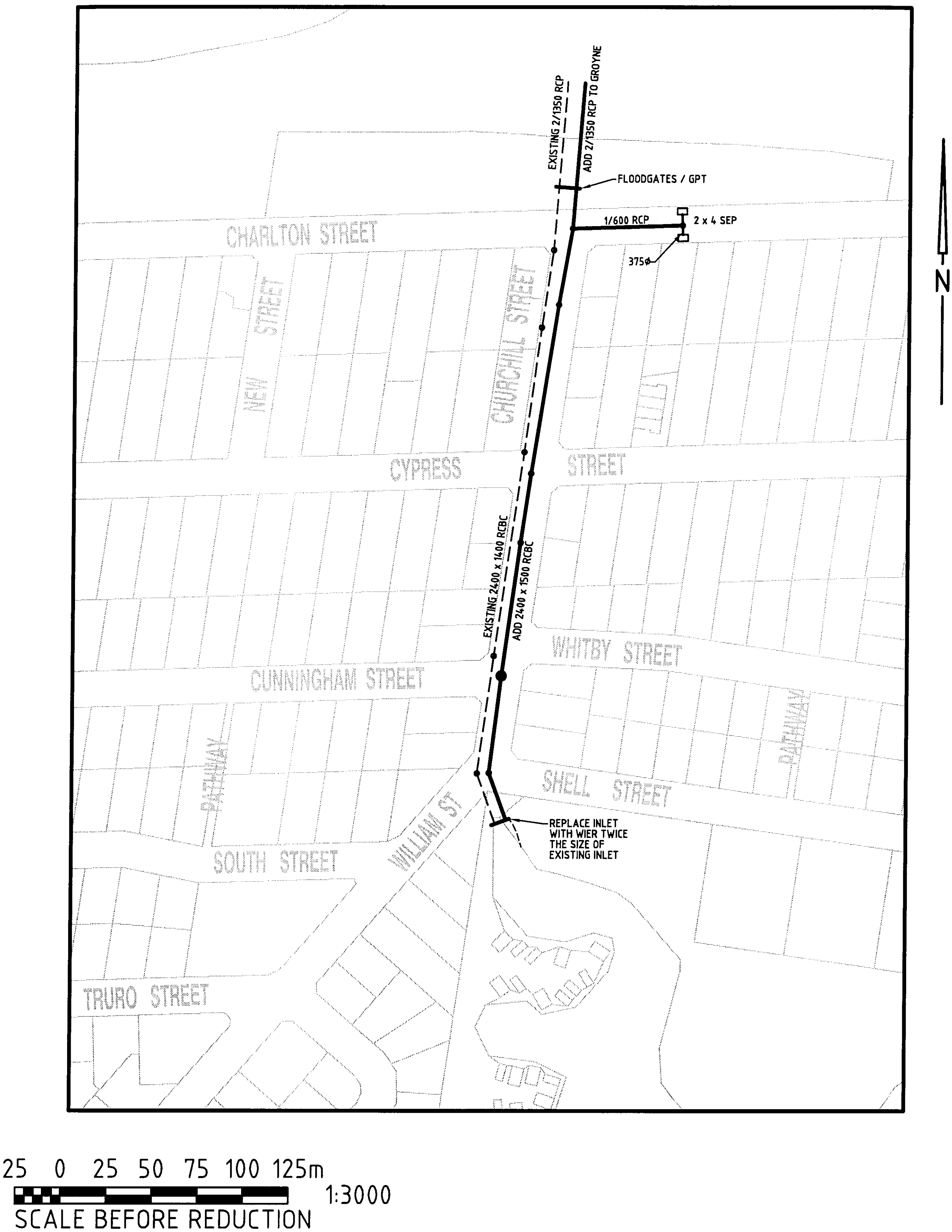

Rev: Orig. Date: 22 February 2002

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CHURCHILL ST. AUGMENTATION

FIGURE 7

Project No.: 2919/20
CAD FILE: I:\2919-20\acad\Final Report\FIGURE-7.dwg
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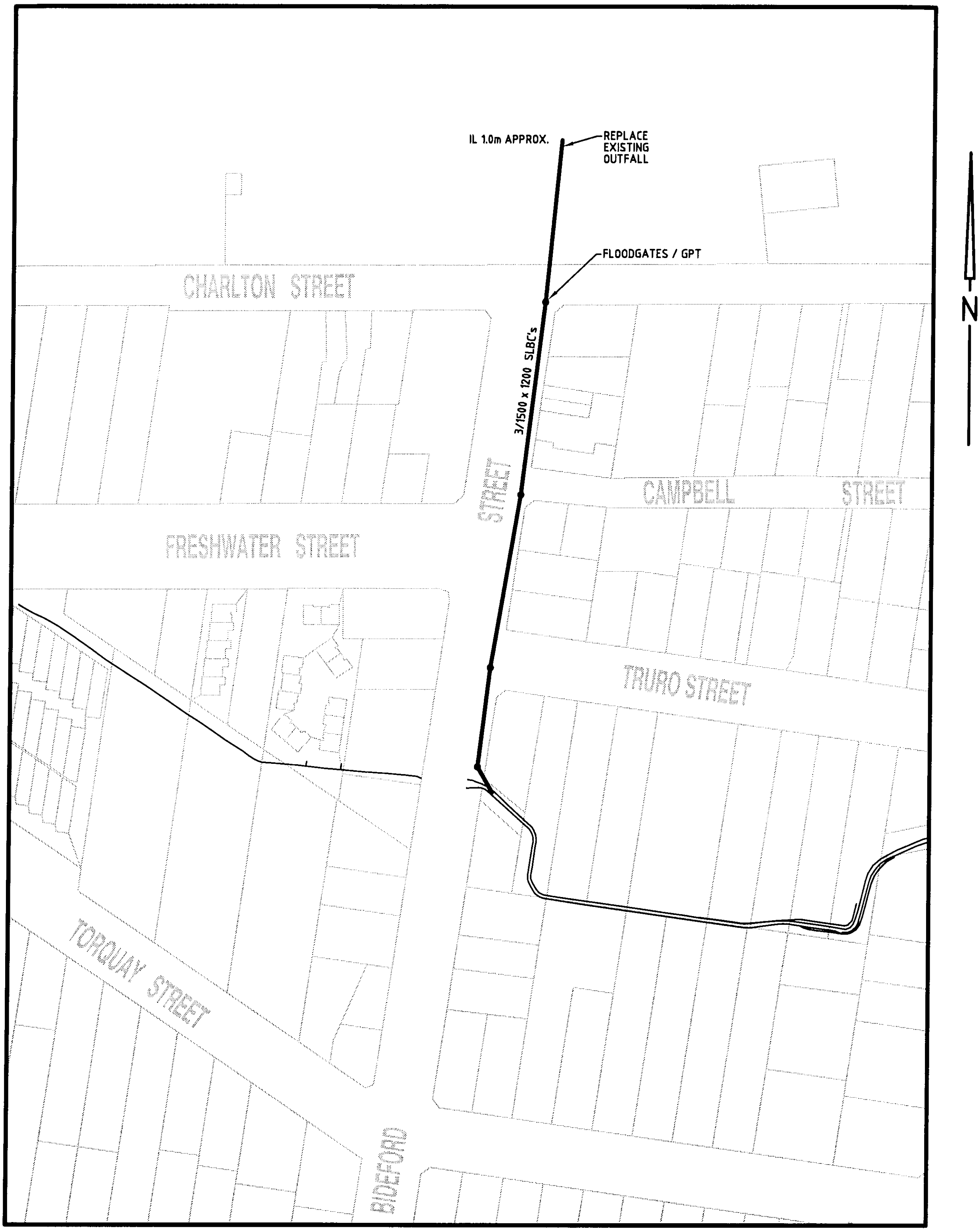
LOWLANDS LAGOONS DRAINAGE STUDY
HERVEY BAY CITY COUNCIL



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Rev: Orig. Date: 22 February 2002

BIDEFORD ST. AUGMENTATION

FIGURE 8

Project No.: 2919/20

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HERVEY BAY CITY COUNCIL

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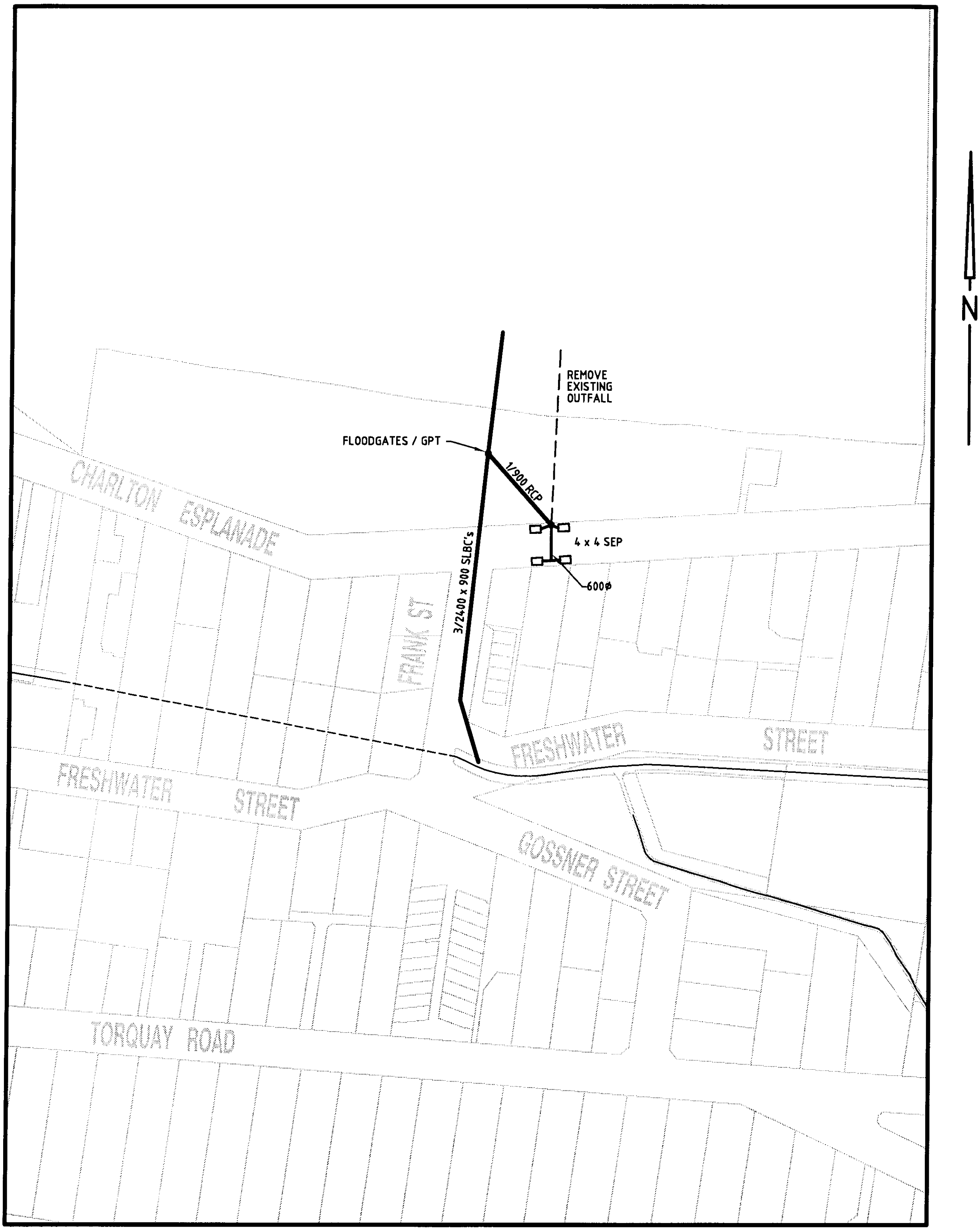

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Rev: Orig. Date: 22 February 2002

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FRANK ST. AUGMENTATION

FIGURE 9

Project No.: 2919/20

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LOWLANDS LAGOONS DRAINAGE STUDY
HERVEY BAY CITY COUNCIL

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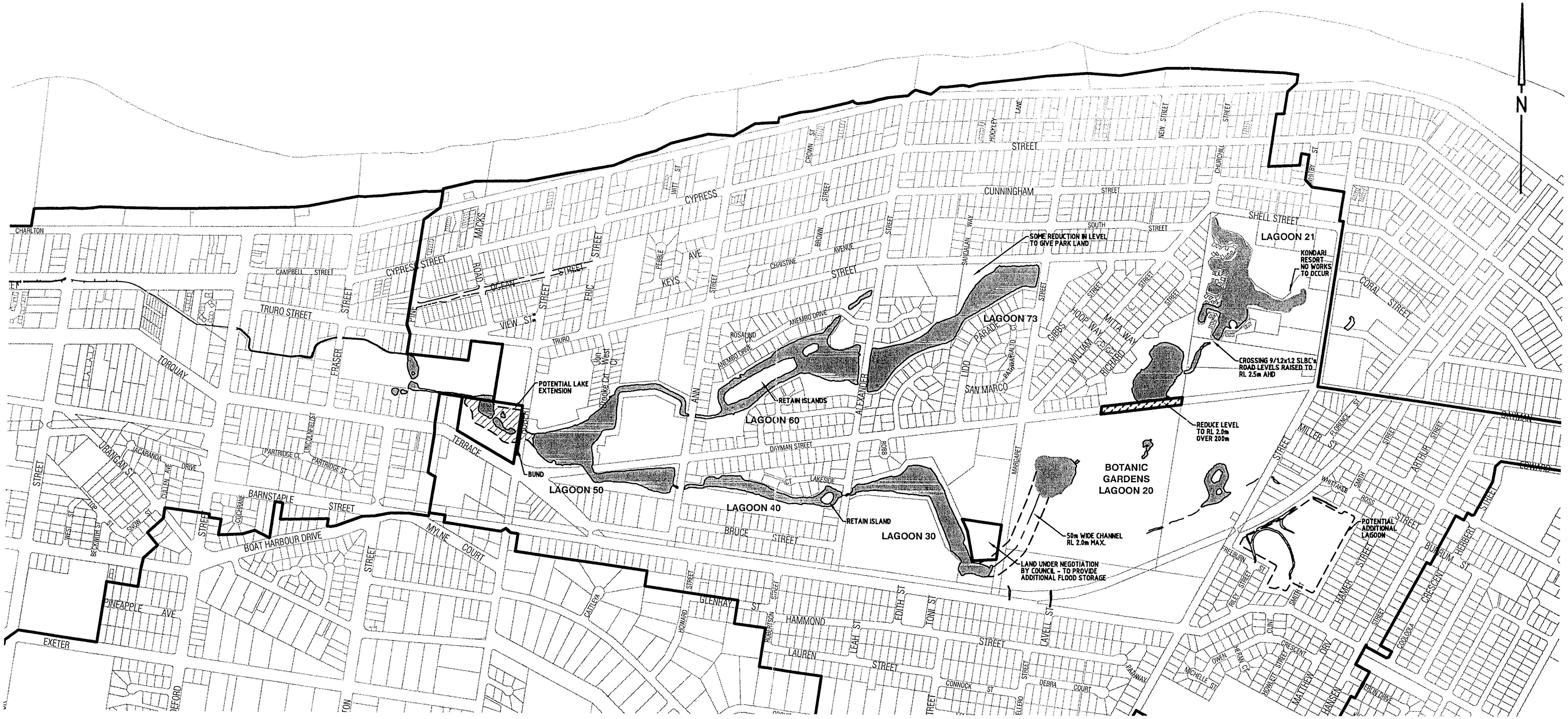

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LAGOON IMPROVEMENT OVERVIEW

FIGURE 10

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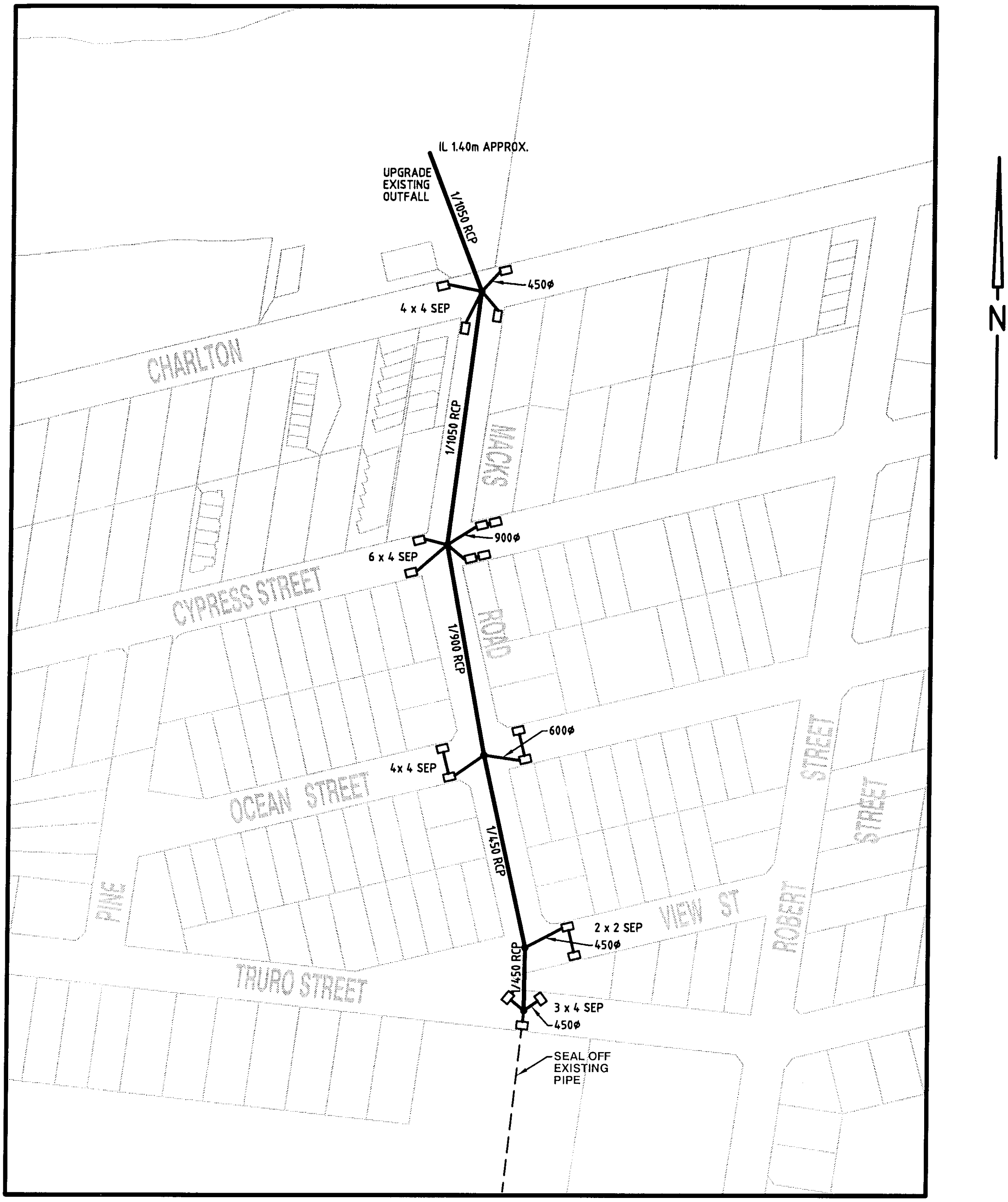

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MACKS RD. RELIEF DRAINAGE WORKS

OPTION A - DRAIN TO OCEAN

FIGURE 11

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-11.dwg
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HERVEY BAY CITY COUNCIL

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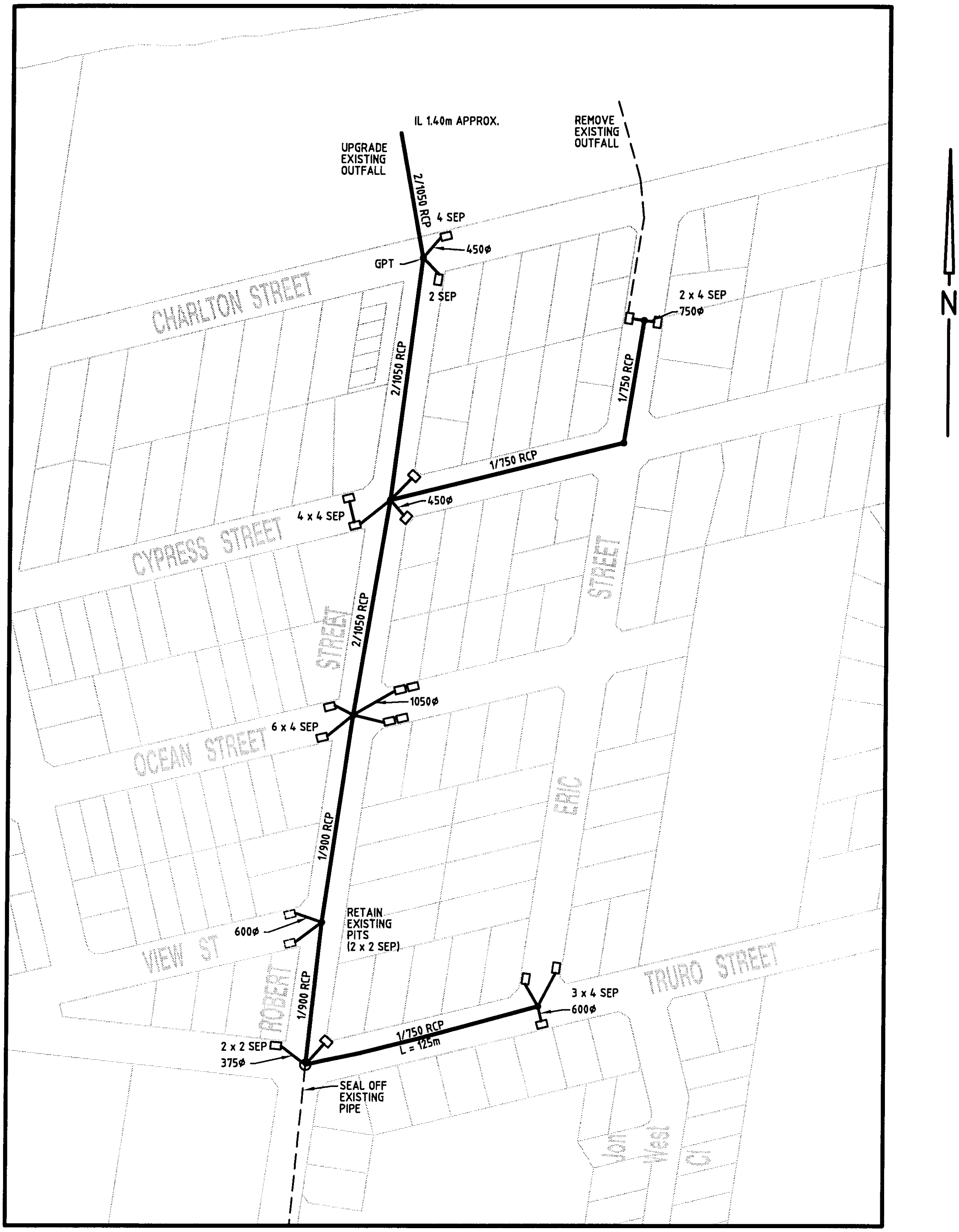

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ROBERT ST. DRAINAGE WORKS

OPTION A - DRAIN TO OCEAN

FIGURE 12

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-12.dwg
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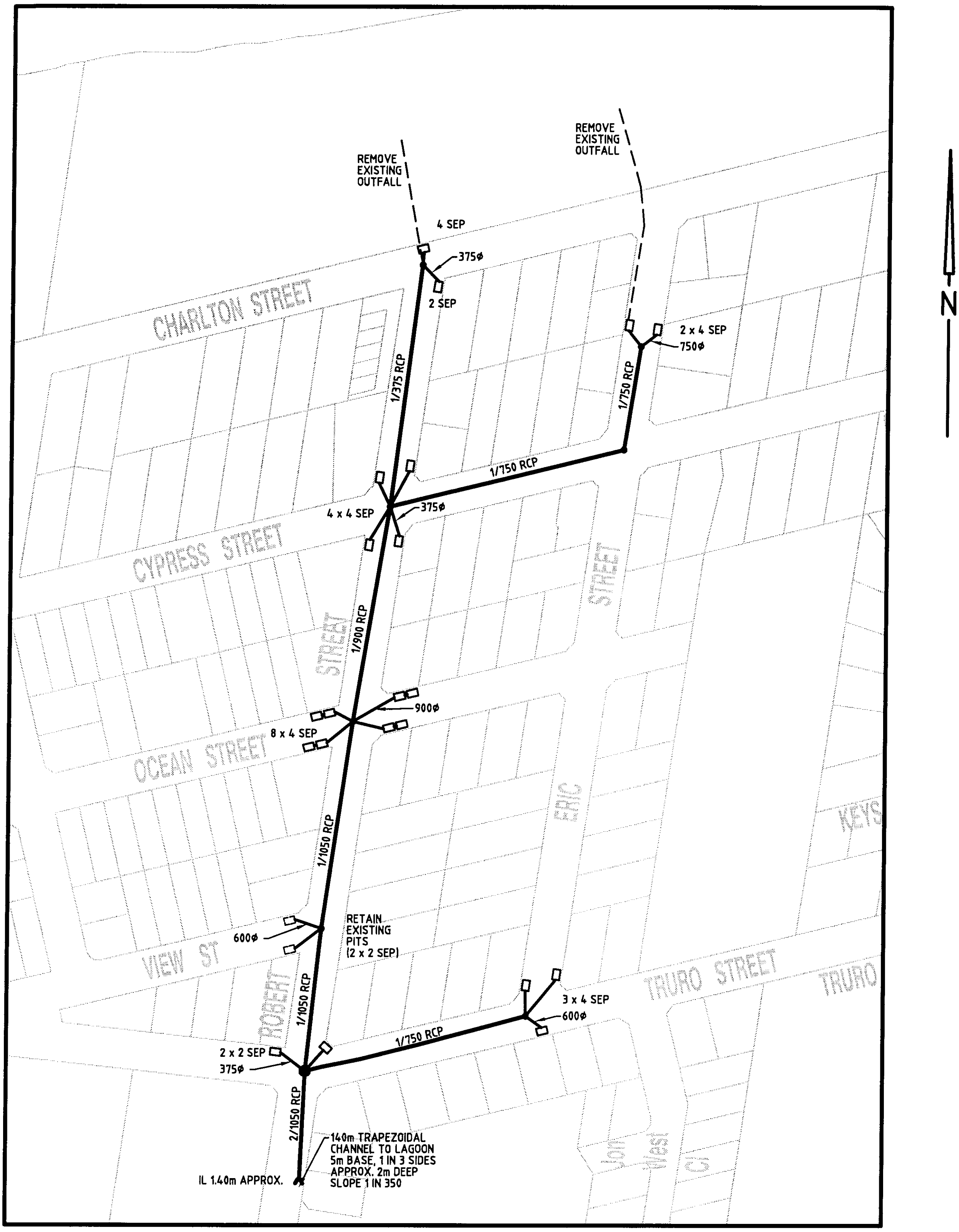
LOWLANDS LAGOONS DRAINAGE STUDY
HERVEY BAY CITY COUNCIL



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ROBERT ST. RELIEF DRAINAGE WORKS

OPTION B1 - DRAIN TO LAGOONS

MINOR EVENT SOLUTION

FIGURE 13

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-13.dwg
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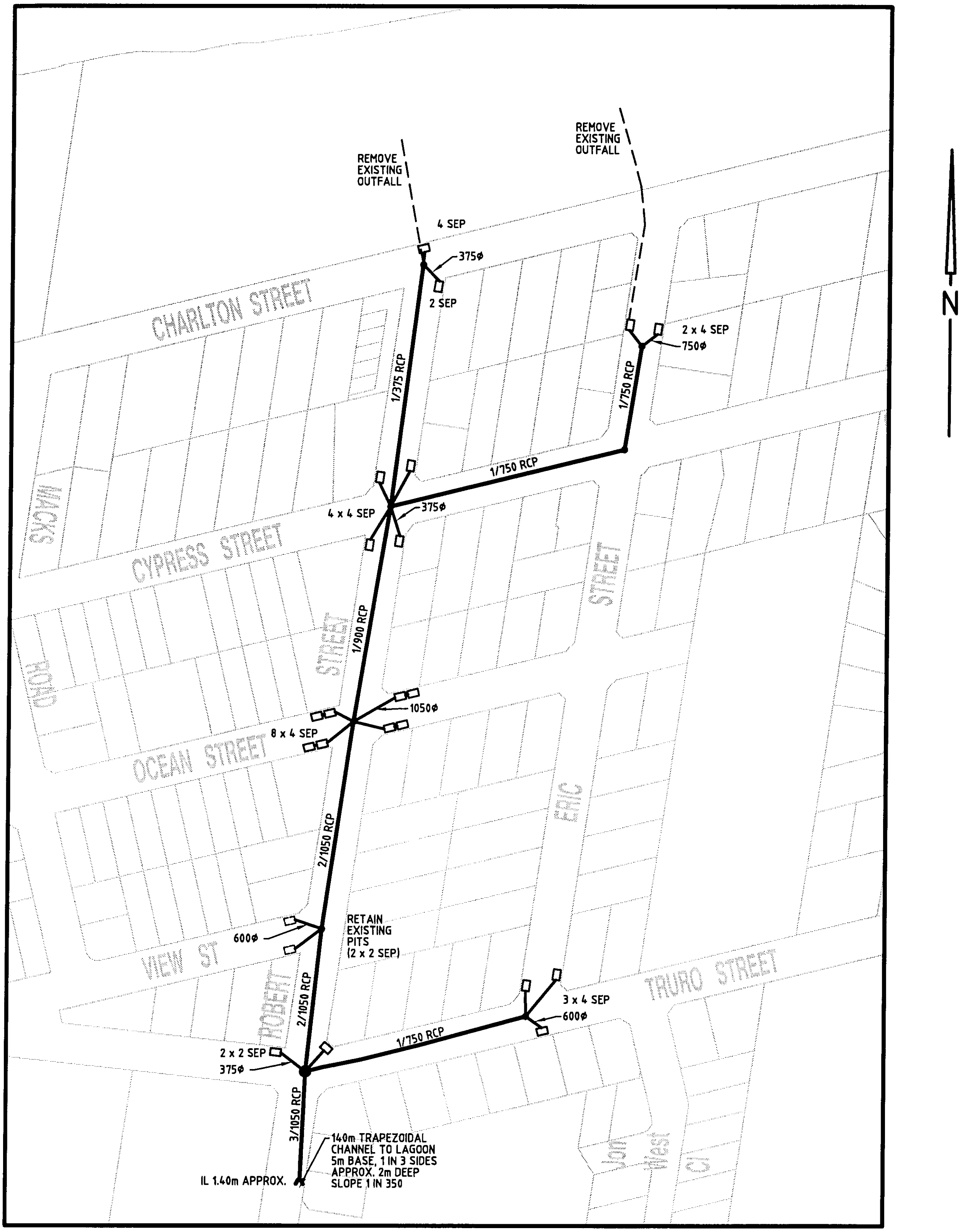
LOWLANDS LAGOONS DRAINAGE STUDY
HERVEY BAY CITY COUNCIL

Cardno MBK



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ROBERT ST. RELIEF DRAINAGE WORKS

OPTION B2 - DRAIN TO LAGOONS

MAJOR EVENT SOLUTION

FIGURE 14

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-14.dwg
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LOWLANDS LAGOONS DRAINAGE STUDY

HERVEY BAY CITY COUNCIL

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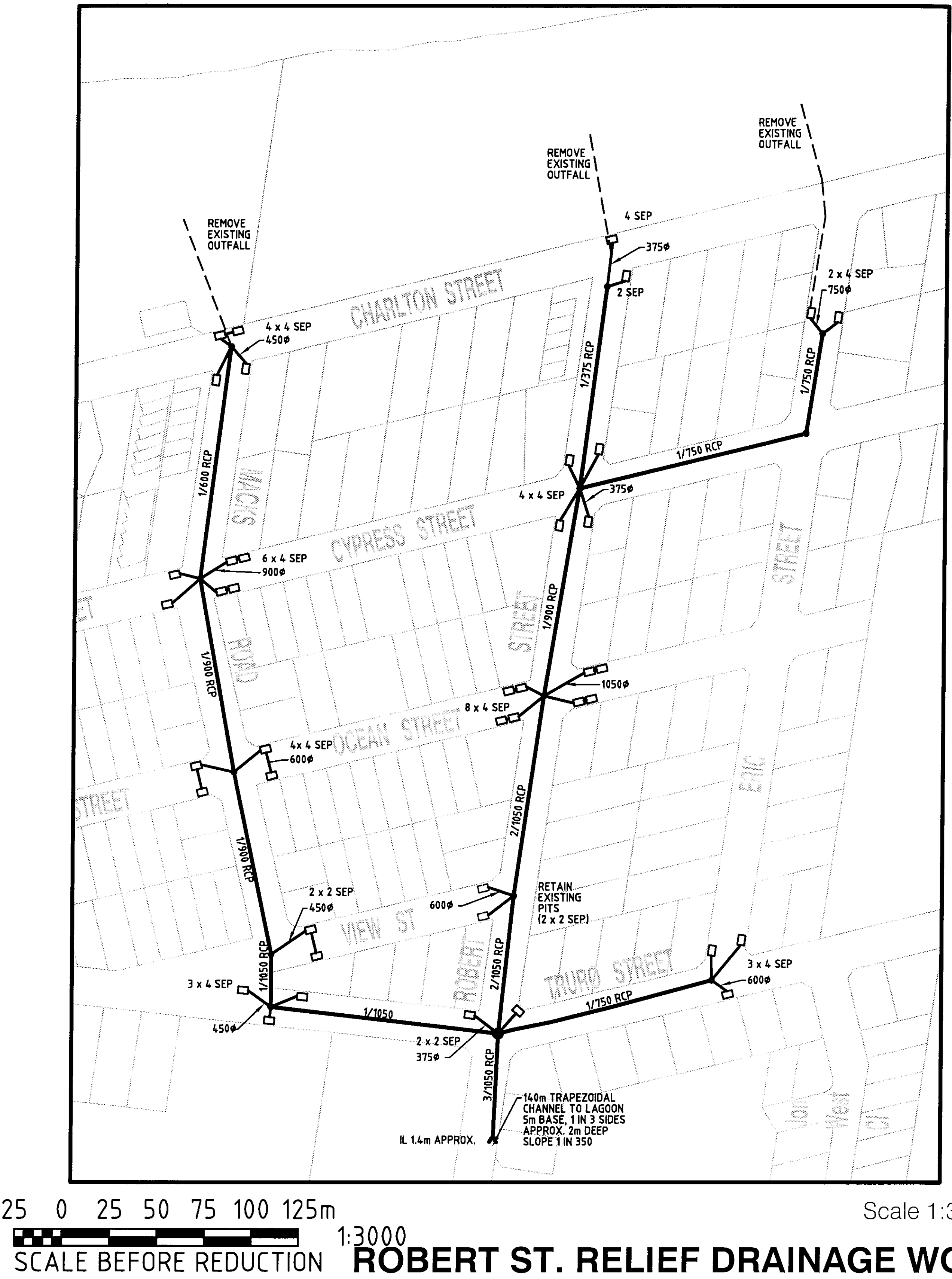

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ROBERT ST. RELIEF DRAINAGE WORKS

OPTION C - DRAIN ROBERT ST. & MACKS RD. TO LAGOONS

Rev: Orig. Date: 22 February 2002

FIGURE 15

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-15.dwg
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HERVEY BAY CITY COUNCIL

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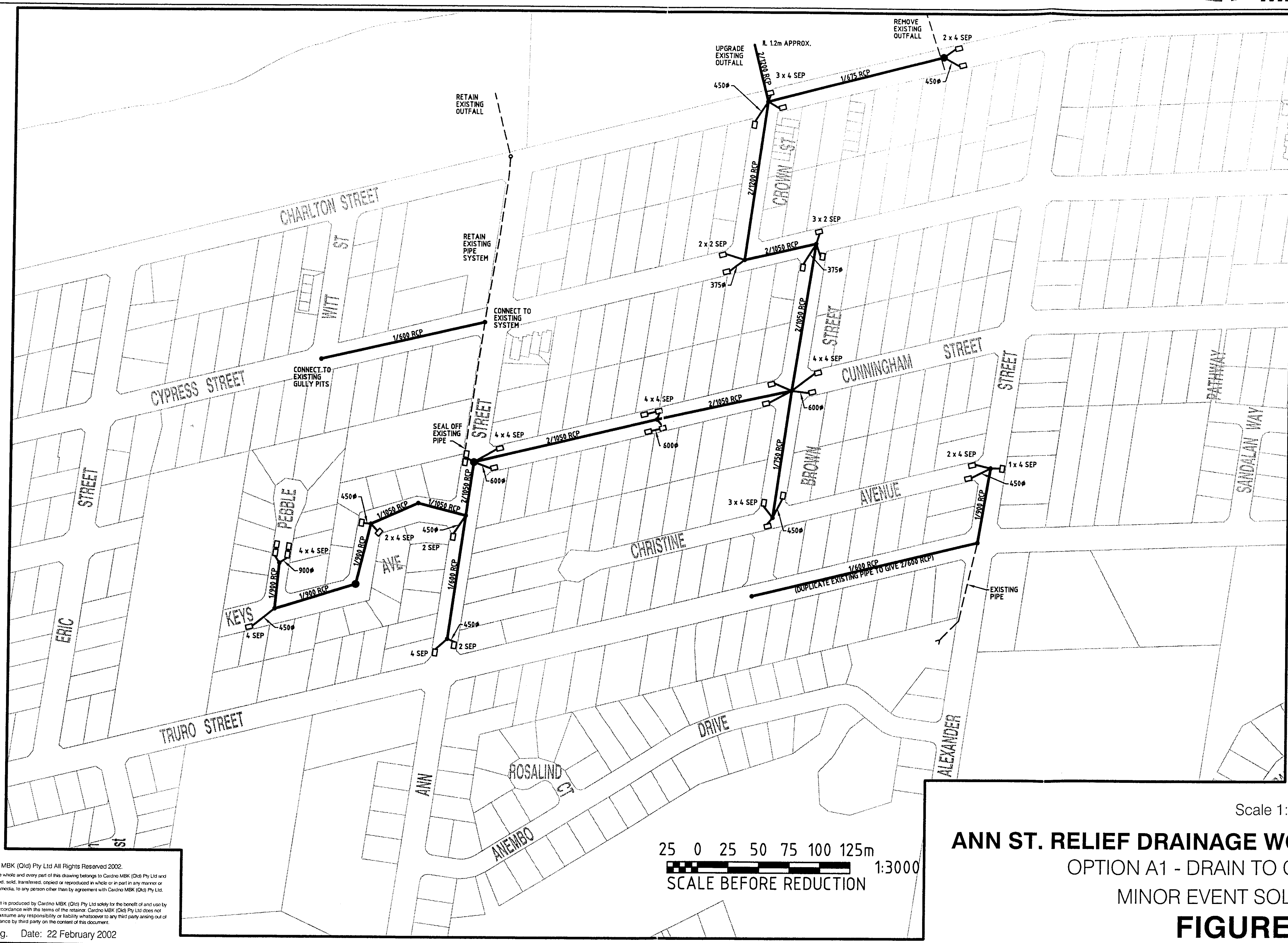

Rev: Orig. Date: 22 February 2002

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ANN ST. RELIEF DRAINAGE WORKS

OPTION A1 - DRAIN TO OCEAN

MINOR EVENT SOLUTION

FIGURE 16

Project No.: 2919/20

CAD FILE: h:\2919-20\acad\Final Report\FIGURE-16.dwg
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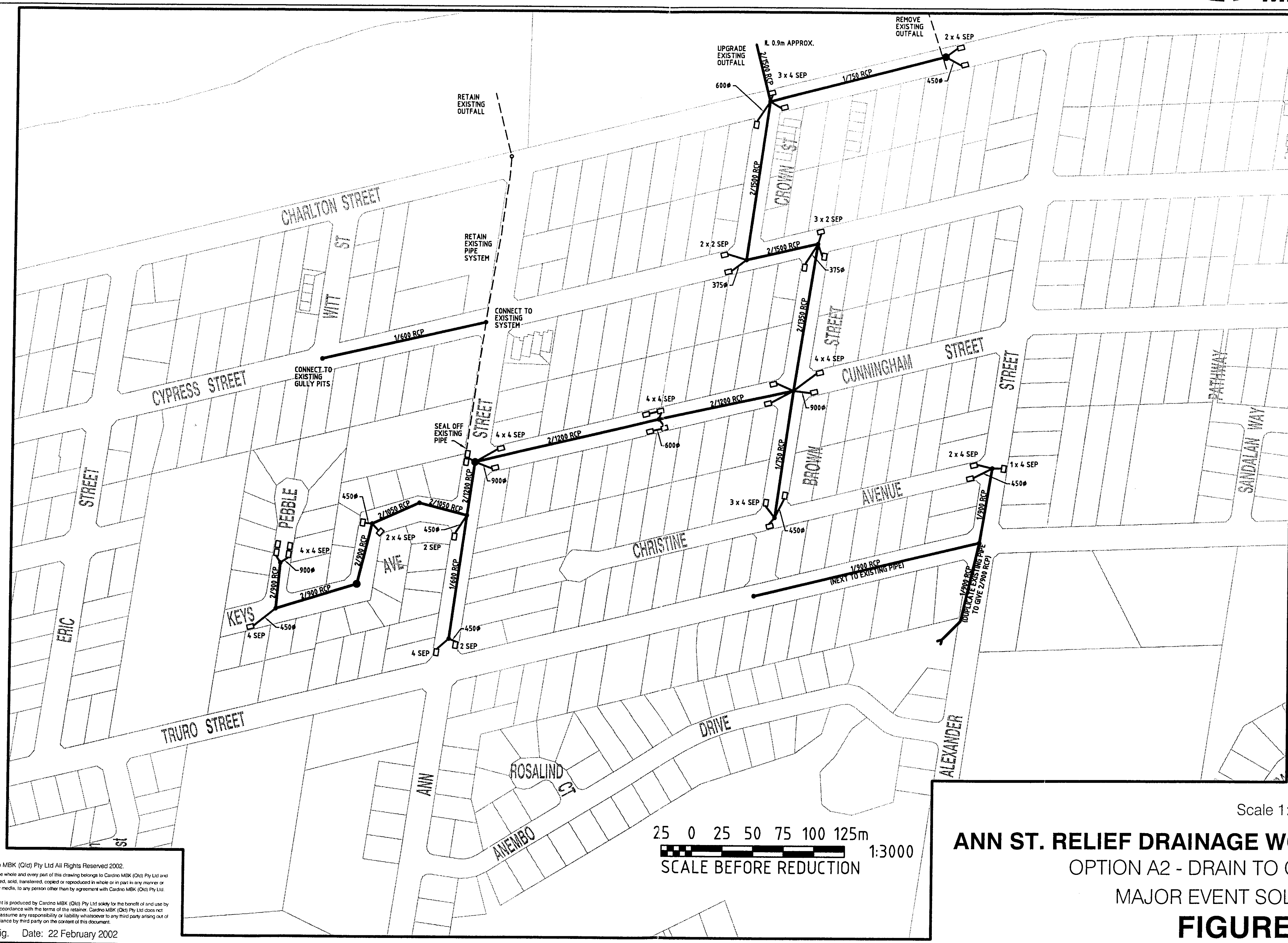
Cardno MBK



Scale 1:3000 (A3)

ANN ST. RELIEF DRAINAGE WORKS

OPTION A2 - DRAIN TO OCEAN

MAJOR EVENT SOLUTION

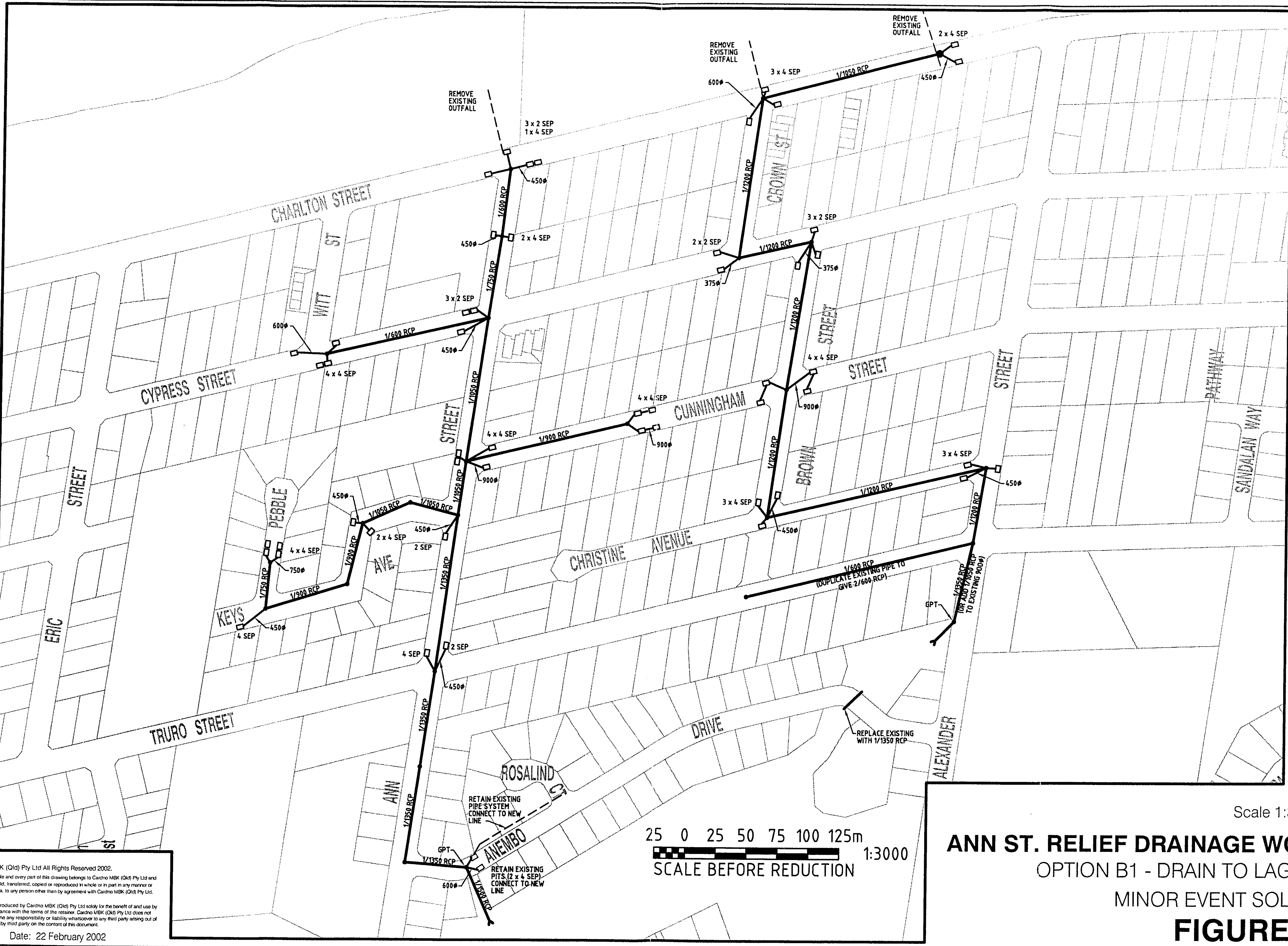
FIGURE 17

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-17.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 12 November, 2003 - 2:55pm

Cardno MBK



Scale 1:3000 (A3)

ANN ST. RELIEF DRAINAGE WORKS

OPTION B1 - DRAIN TO LAGOONS

MINOR EVENT SOLUTION

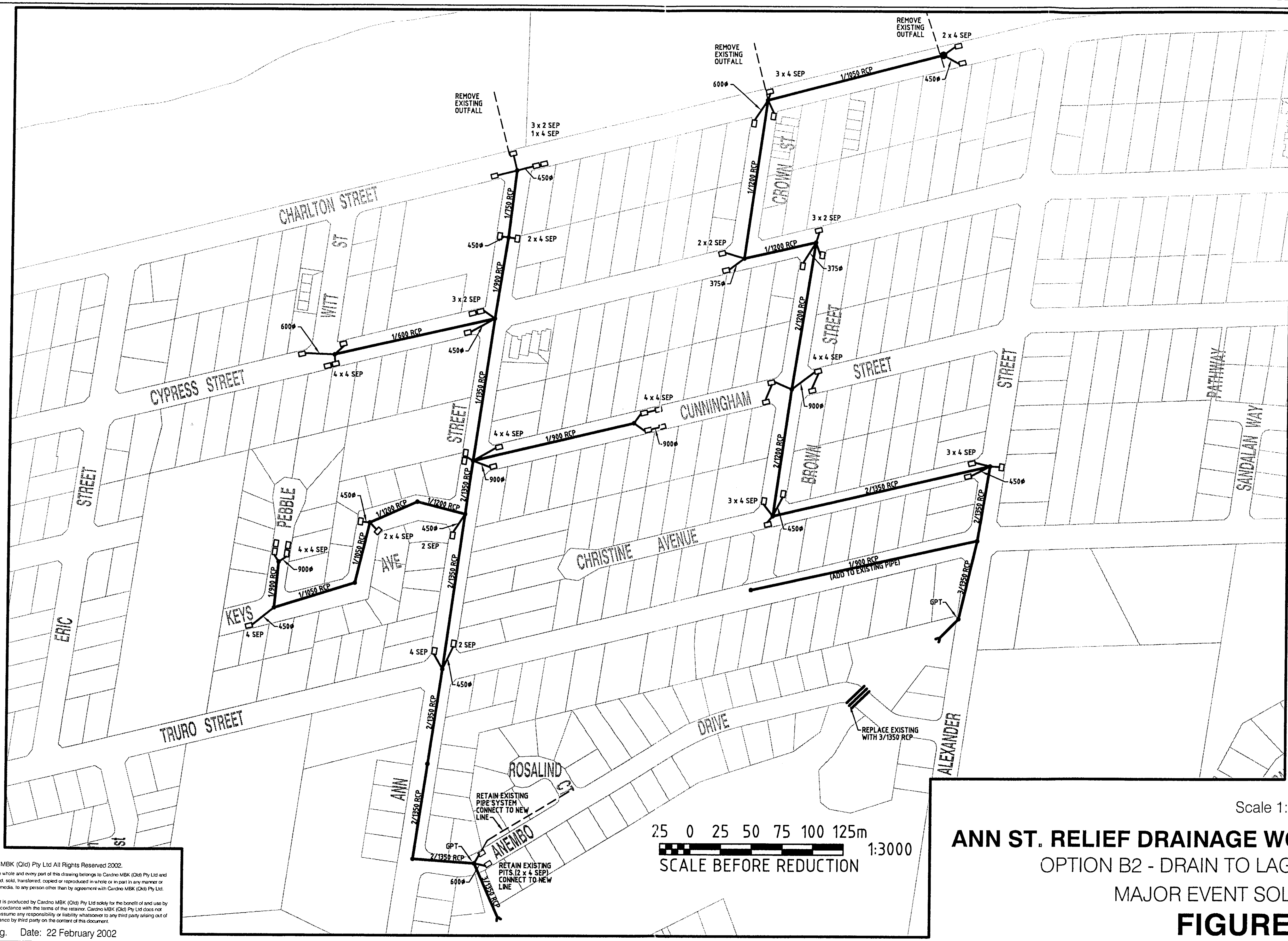
FIGURE 18

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-18.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 12 November, 2003 - 3:01pm

Cardno MBK



Scale 1:3000 (A3)

ANN ST. RELIEF DRAINAGE WORKS

OPTION B2 - DRAIN TO LAGOONS

MAJOR EVENT SOLUTION

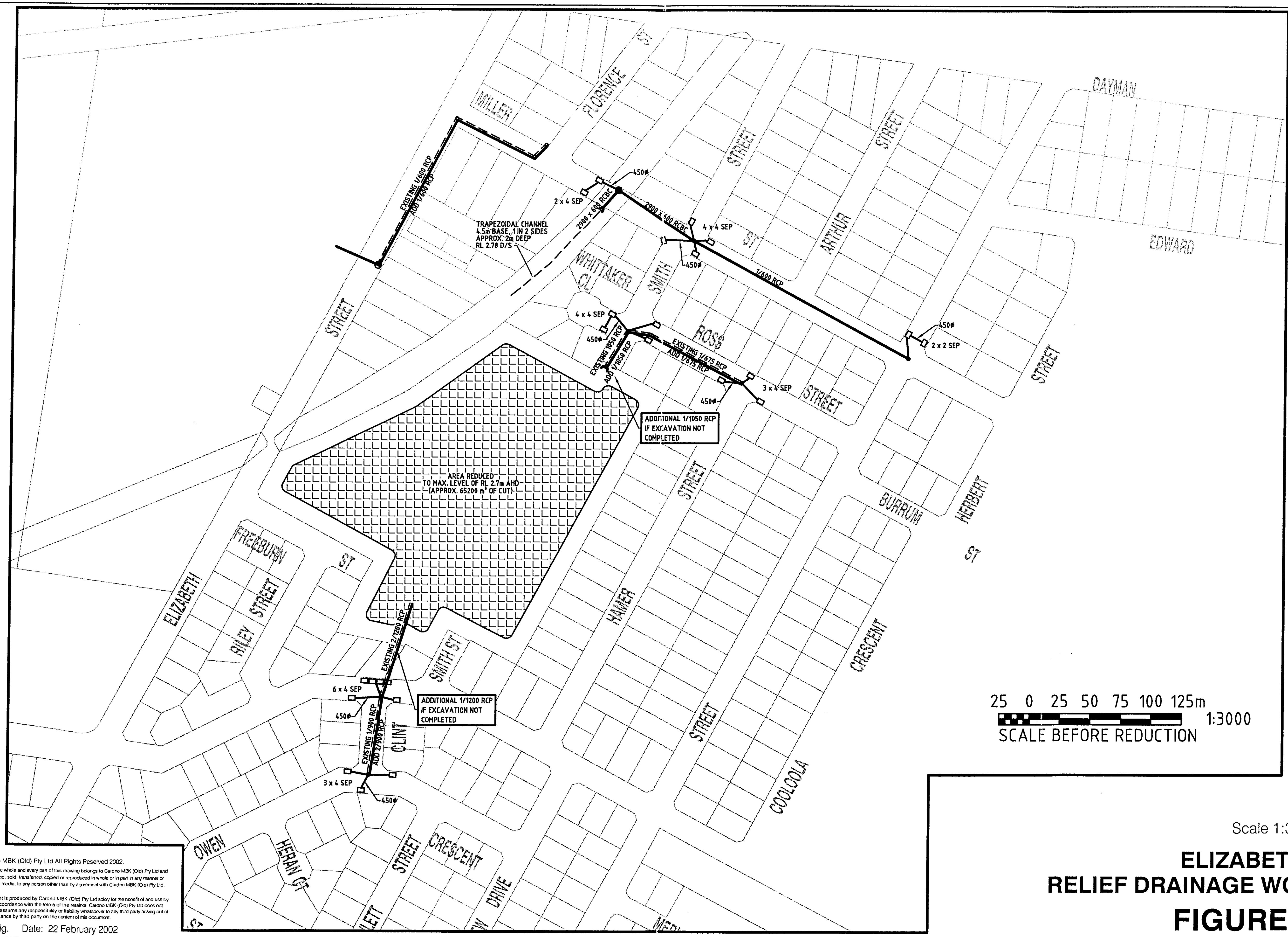
FIGURE 19

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-19.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 12 November, 2003 - 3:01pm

Cardno MBK



Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

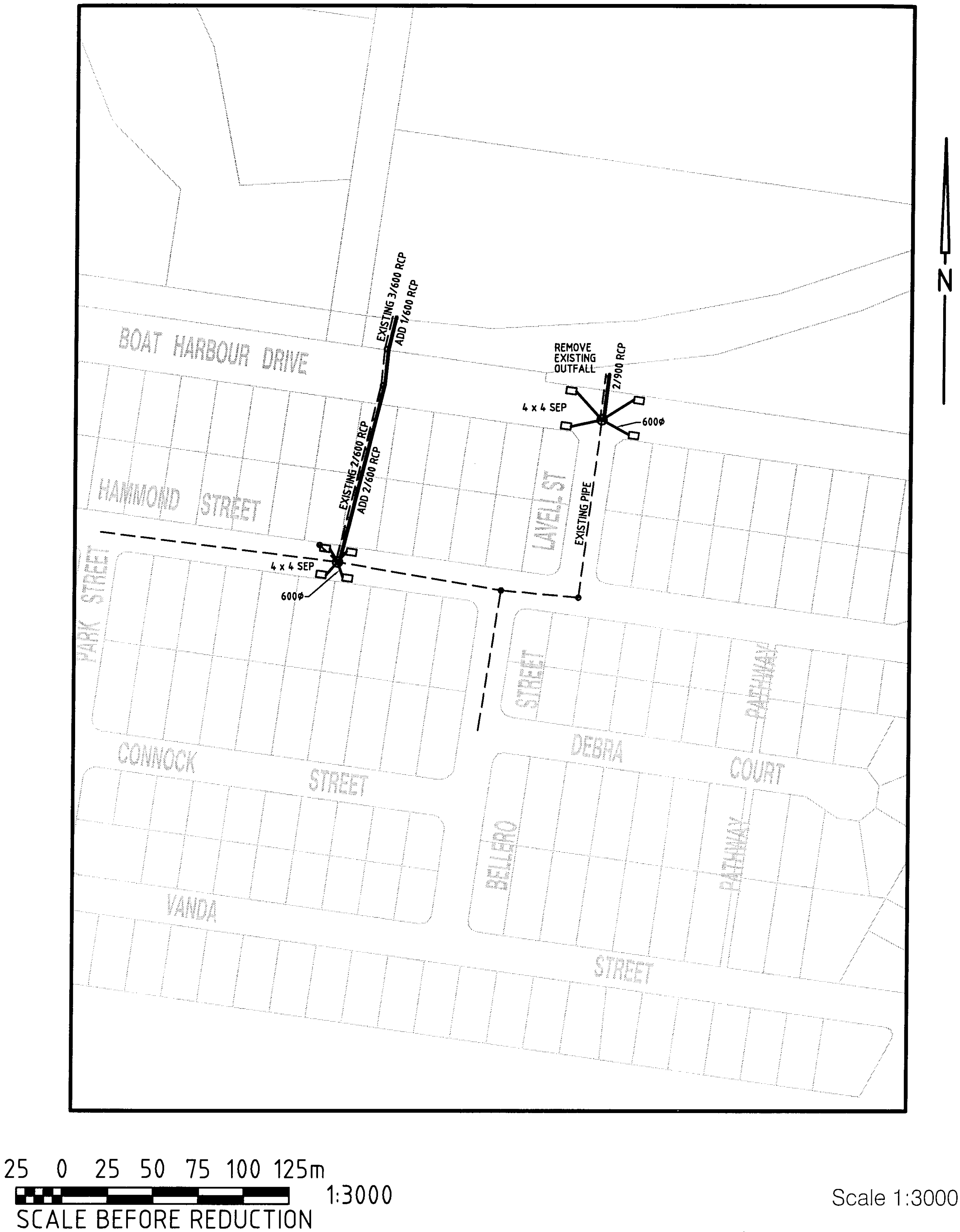
CAD FILE: I:\2919-20\acad\Final Report\FIGURE-20.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 12 November, 2003 - 3:09pm

Scale 1:3000 (A3)

ELIZABETH ST. RELIEF DRAINAGE WORKS

FIGURE 20

Cardno MBK



Scale 1:3000 (A4)

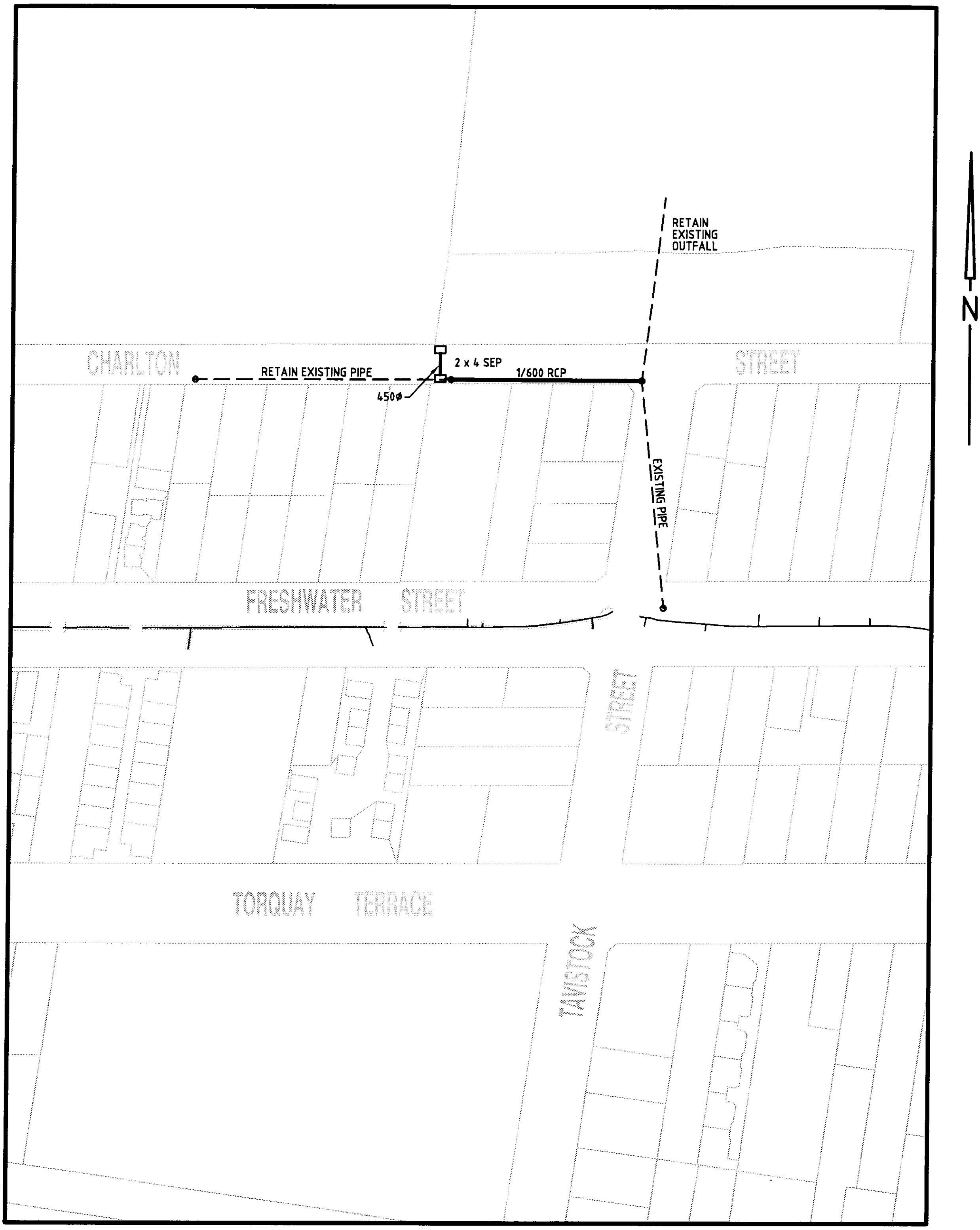
HAMMOND ST. / LAVELL ST.
RELIEF DRAINAGE WORKS
FIGURE 21

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-21.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 11 November, 2003 - 6:20pm

Cardno MBK



25 0 25 50 75 100 125m

1:3000

SCALE BEFORE REDUCTION

Rev: Orig. Date: 22 February 2002

Scale 1:3000 (A4)

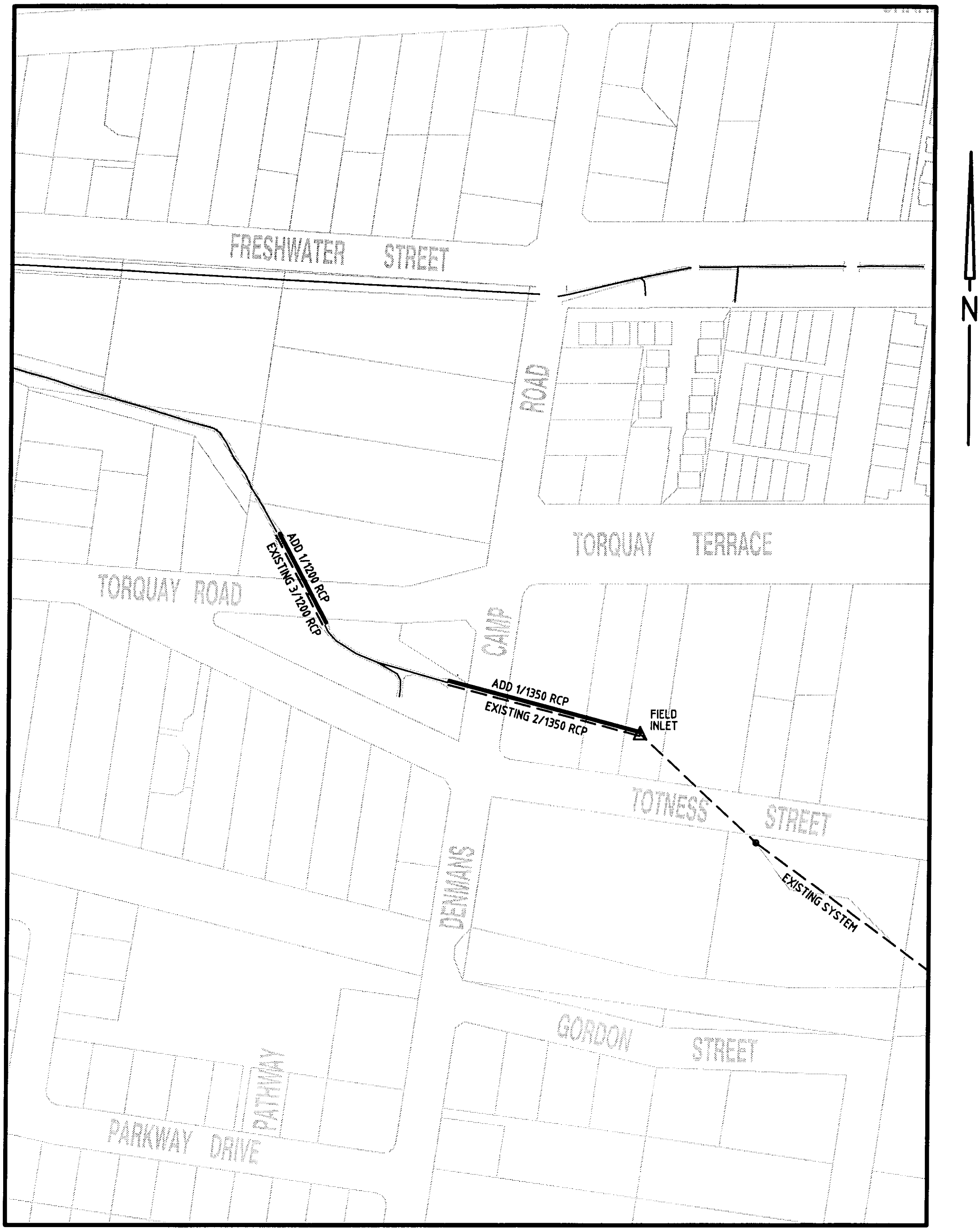
TAVISTOCK ST. AUGMENTATION

FIGURE 22

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-22.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 11 November, 2003 - 6:20pm

LOWLANDS LAGOONS DRAINAGE STUDY
HERVEY BAY CITY COUNCIL



25 0 25 50 75 100 125m

1:3000

SCALE BEFORE REDUCTION

Scale 1:3000 (A4)

TORQUAY RD. / DENMANS CAMP RD.

RELIEF DRAINAGE WORKS

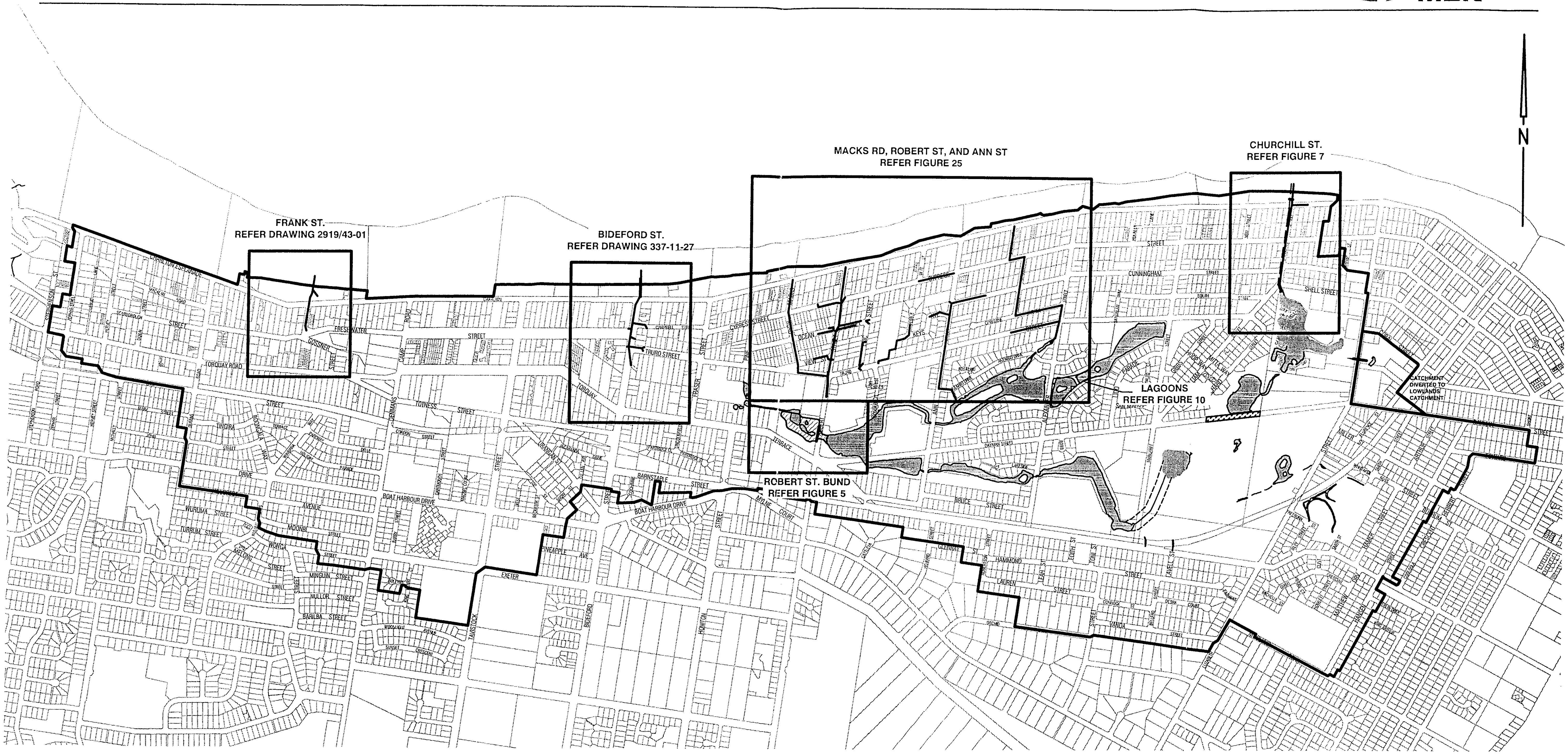
FIGURE 23

Rev: Orig. Date: 22 February 2002

Project No.: 2919/20

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-23.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 11 November, 2003 - 6:20pm

Cardno MBK



150 0 150 300 450 600 750m

1:15000

SCALE BEFORE REDUCTION

Rev: Orig. Date: 21 December 2000

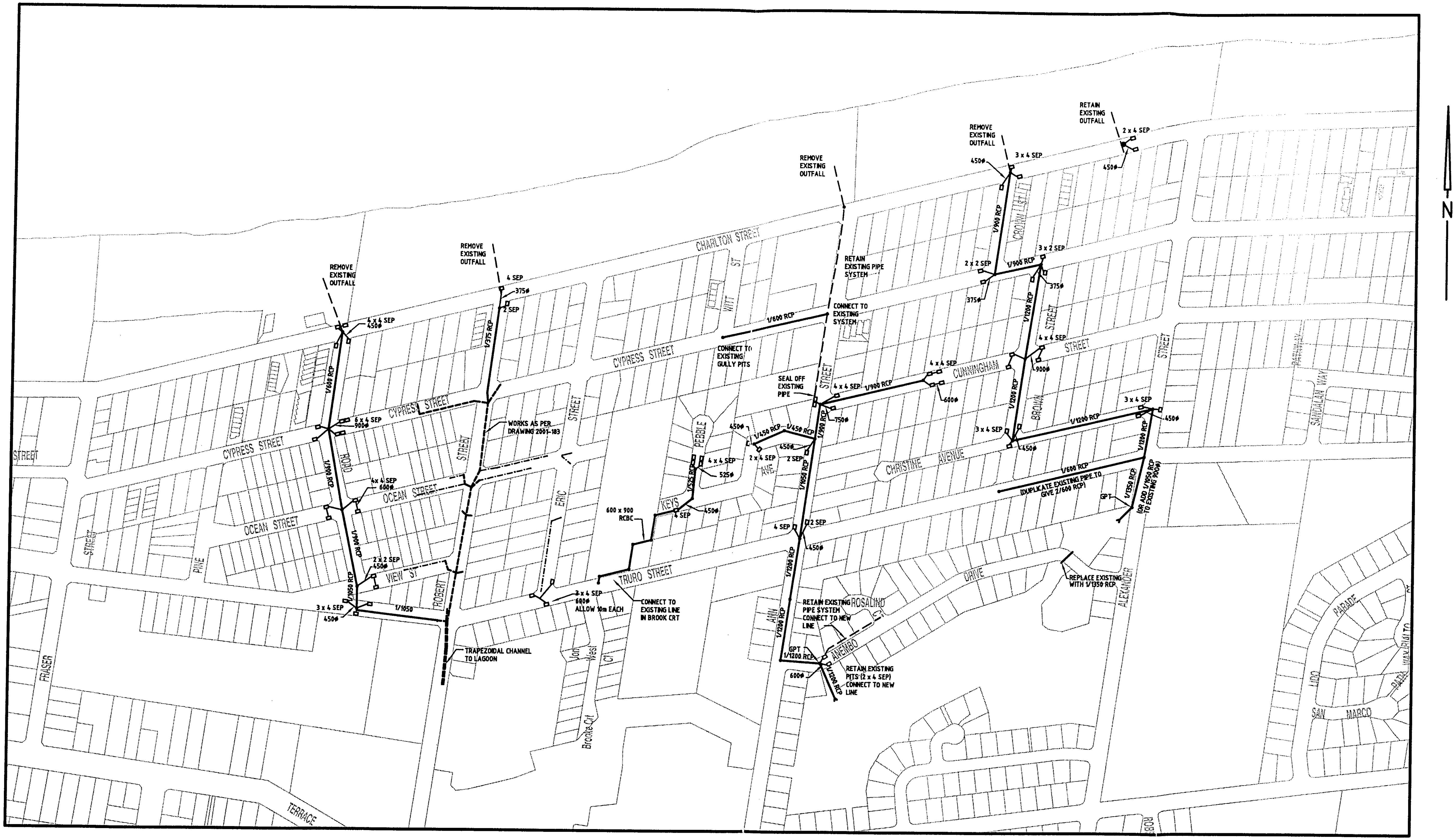
Scale 1:15000 (A3)

ADOPTED RELIEF DRAINAGE WORKS

KEY PLAN

FIGURE 24

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-24.dwg
XREF's: Locality
DATE PLOTTED: 12 November, 2003 - 3:10pm



50 0 50 100 150 200 250m

SCALE BEFORE REDUCTION

1:5000

Rev: Orig. Date: 04 November 2003

Project No.: 2919/20/05

CAD FILE: I:\2919-20\acad\Final Report\FIGURE-25.dwg
XREF's: xr-udd-map with dcdb; xr-drainage-ult
DATE PLOTTED: 12 November, 2003 - 3:14pm

NOTE: Drawing 2001-183 works shown:

- - - - - - - INITIAL WORKS

-·-·-·-·-·- FUTURE WORKS

Scale 1:5000 (A3)

ADOPTED RELIEF DRAINAGE WORKS

MACKS ROAD TO ALEXANDER STREET

FIGURE 25