BORNHORST + WARD
CONSULTING ENGINEERS
CIVIL AND STRUCTURAL



# MASTERPLANNING CIVIL ENGINEERING REPORT

FOR THE PROPOSED
NEIGHBOURHOOD DEVELOPMENT

LOCATED AT
DALEYS ROAD AND BINNIES ROAD
DEEBING HEIGHTS

Lots 3, 5 & 6 on RP180932 and
Lots 328 & 329 on S3157

PREPARED FOR
BACKUP AUSTRALIA PTY LTD
&
P&D HOLDINGS PTY LTD

Bornhorst + Ward Pty Ltd
Level 4, 67 Astor Terrace, Spring Hill, Qld 4000
Ph. (07) 3013 4699 / mail@bornhorstward.com.au
A.B.N. 78 010 151 354

A.B.N 78 010 151 354
A.C.N 010 151 354
Level 4, 67 Astor Terrace
Spring Hill QLD 4000
Phone: (07) 3013 4699
Fax: (07) 3013 4611
E-mail: mail@bornhorstward.com.au

Bornhorst & Ward Project No. **08199**

If you have any queries regarding this report please contact: Blair Batts

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## 1. PROJECT DESCRIPTION

Bornhorst and Ward Consulting Engineers Pty Ltd have been commissioned by Backup Australia Pty Ltd and P&D Holdings Pty Ltd to undertake an investigation of the civil engineering requirements pertaining to the proposed subdivision located on Daleys Road, Deebing Creek.

The proposed development layout is shown on Land Partners drawing number BRSS5365.0001-001D attached in Appendix A of this document.

The proposed development will consist of the construction of a high density residential sub-division creating 426 individual allotments with associated open space and road network. Individual allotment vary in size between 250 and >600m$^2$.

Reporting undertaken here within will address the master planning issues of stormwater discharge quantity/quality, sewerage, water supply, road pavements and earthworks.

This report is prepared in support of the ROL and Development Applications for the proposed development.

## 2. SITE LOCATION AND DESCRIPTION

The proposed development site is described as Lots 3, 5 and 6 on RP180932 and Lots 328 and 329 on S3157 located on Daleys Road, Deebing Creek. The site is bounded by Daleys Road to the east, Deebing Creek to the west and the Cunningham Highway to the north.

The property is currently undeveloped and the majority of the site area can be considered as moderate bushland with pockets of sparse bush and grasses. The site generally slopes towards the north east and north west with natural elevations ranging between 30 - 72m AHD with relatively steep topography throughout.

Deebing Creek aligns with the western boundary of the property; however the natural site storm runoff is conveyed through two minor natural gullies which traverse the site from south to north, crossing under the Cunningham Highway at the north east and North West corners of the development site.

## 3. EXISTING CIVIL INFRASTRUCTURE

Existing infrastructure within the development is minimal due to the relatively isolated location and steep topography. There is currently no formed road access to the site, the nearest road networks are located approximately 750m south west of the development site at the intersection of Binnies Road /South Deebing Creek Road/Deebing Creek Connection Road.

The predominant water supply servicing the development area comes from the existing 600mm dia water main currently constructed within the existing Binnies Road, Road Reserve extending from the intersection of South Deebing Creek Road/Binnies Road and Deebing Creek Connection Road, east along Binnies Road and continues further east past the future intersection of Binnies Road and Daleys Road. Further 300mm dia water infrastructure is located within the north west corner of the development and a 450mm dia main is located within the Daleys Road Reserve to the east.

A 600mm diameter trunk sewer main currently traverses the western corner of the site, grading to the north, under the Cunningham Highway. A 300mm dia stub has been constructed off this existing main on the eastern side of Deebing Creek to service future development.

Stormwater runoff from the development is generally conveyed through the site via two natural gullies which discharge to under the Cunningham Highway to the north via concrete pipe culverts.

## 4. PROPOSED CIVIL WORKS AND INFRASTRUCTURE

The following sections consider stormwater, potable water, sewerage and road infrastructure required to service the proposed development. Consideration of constraints imposed by the capacity/availability of existing infrastructure has been undertaken.

## 5. STORMWATER MANAGEMENT

### 5.1 INTRODUCTION

In considering the disposal system for stormwater from the development site, the following constraints and guidelines have been taken into account:-

- Ripley Valley Master Planned Area Structure Plan;
- ULDA Street and Movement Network (Draft) Guidelines;
- Healthy Waterways Water Sensitive Urban Design Guidelines – Technical Design; Guidelines for South East Queensland;
- Ipswich City Council Planning Scheme – Design Standards for Stormwater Drainage;
- BCC Draft Guidelines for Pollutant Export Modelling in Brisbane Version 7; and
- Impact on the environmental values of neighbouring waterways.

To satisfy the guidelines indicated above and provide designs in accordance with Best Management Engineering Practice, the system must control stormwater quality, quantity and velocity.

Water sensitive urban design (WSUD) methods as referenced from Ipswich City Council, along with regional guidelines (Healthy Waterways), will also be adopted to control volume and velocity of flow as well as satisfying water quality objectives (WQOs). These methods will include the use of bio-retention areas and on site rainwater harvesting which will result in stormwater being both detained and treated accordingly before discharging from the site.

#### 5.1.1 Existing Site Conditions

The majority of the proposed development site is currently rural, containing a mixture of dense and sparse vegetation coverage.

The site area can generally be divided into two main catchment areas, A and B, both containing relatively steep topography and discharging to the north.

Catchment 'A' extends through the proposed development site and approximately 1.5Km to the south east. The catchment comprises 93.4Ha of generally rural bush and farming land of which the proposed development site contributes 9.56Ha of the total area. Catchment 'A' drains to a natural gully which flows in a northerly direction, through the north east corner of the proposed development site and discharges under the Cunningham Highway via 4 x 1650mm concrete pipe culverts.

Catchment 'B' comprises 19.5Ha of generally medium to dense bush land. The catchment extends to the site boundary to the south and is defined by the two ridge lines running north south within the development area. Storm runoff from the catchment is conveyed via a natural gully which flows in a northerly direction, through the north west corner of the proposed development site and discharges under the Cunningham Highway under the Highway via a single 1650mm concrete pipe culvert.

## 5.2 STORMWATER QUANTITY MANAGEMENT

The proposal detailed in this report is for the construction of a residential neighbourhood consisting of T3 suburban and T4 urban areas, service and neighbourhood roads, associated open space, drainage reserves and infrastructure.

It is anticipated that the resulting development will increase the impervious area of the catchment to approximately 80% of the total area and thus a significant localised increase in stormwater runoff and peak discharges will result.

This report will consider the effects of the increase runoff from the development on the developed catchment and adjoining properties, including the existing culverts under the Cunningham Highway.

Mitigation measures will be proposed to reduce both the peak discharge rates and volumes at the nominated legal point of discharge.

An electronic hydrologic and hydraulic analysis of the existing and proposed development has been undertaken to determine the effects of the development of the surrounding environment and to size mitigation measures within the proposed development.

Existing hydraulic data exists for Deebing Creek to the west of the proposed development, however it is not the intention of this development to encroach below the existing 1 in 100 year ARI flood level within this system, nor is it intended that stormwater runoff from the developed catchment will discharge directly to Deebing Creek. Therefore no further analysis of this system has been undertaken at this time.

### 5.2.1 Hydrologic Analysis

No existing flow gauge data has been obtained at the Cunningham Highway culverts at this time. Therefore the peak stormwater runoff was calculated using the Rational Method in accordance with the recommendations and parameters described in the Queensland Urban Drainage Manual 2007 (QUDM)

Calculations have been performed at two locations identified along the northern property boundary of the site. These locations have been labelled 'A' and 'B' as defined on the attached catchment drawing, Appendix B.

The analysis takes into consideration the proposed development site as well as an upstream catchment area of approximately 85Ha which contributes flows to discharge location 'A'.

Catchments have been broken into sub-catchments, and sub-catchments have been assigned appropriate $C_{10}$ values which represent their individual catchment characteristics. The calculation does not include any allowance for detention in on dams or localised depressions which may be present within the catchment areas.

Peak flow rates have been calculated using the rural runoff methods described in Chapter 4, Section 4.06.11 of the Queensland Urban Drainage Manual for calculating the time of concentration through the existing catchments.

The following table represents peak flow rates calculated for the existing site.

**Table 1: Existing Peak Flow Rates (Rational Method)**

| ARI (years) | Existing Flow ($m^3/s$) | |
|---|---|---|
| | Point 'A' | Point 'B' |
| 2 | 7.98 | 1.96 |
| 5 | 11.64 | 2.84 |
| 10 | 14.05 | 3.42 |
| 20 | 17.26 | 4.19 |
| 50 | 22.67 | 5.49 |
| 100 | 26.78 | 6.48 |

### 5.2.2 Hydraulic Modelling

A computerised hydraulic model was created using XP Storm software to represent catchments that contribute to flows exiting the site to the north. For the purpose of this analysis, two separate models have been created for the existing and post development scenarios.

XP Storm is a hydraulic modelling software tool that utilizes detailed hydrograph flow analysis and 1D/2D modelling to provide an effective representation of urban and rural stormwater systems. Its function for the proposed development will be to generate peak flow hydrograph data for both the existing and post development scenarios as well as size detention features proposed within the ultimate development.

Using Australian Rainfall & Runoff (AR&R) hydrological data for the Ripley region, stormwater catchment data described in appendix B, and detailed survey of the catchment, the runoff characteristics and detention storage properties have been determined in the XP Storm hydraulic model.

Electronic copies of the XP Storm model are available upon request.

### 5.2.3 Existing XP Storm Model

The XP Storm model was created with nodes representing each catchment area joined with a series of links allowing discharge from each catchment to be combined in a fully 1D model.

Catchment runoff routing was carried out utilizing the "Laurenson Method" of routing as recommended by the XP Storm user manual.

Catchment characteristics utilized in the above Rational Method calculations such as total catchment area, percentage impervious and catchment slope were entered for each individual node.

Infiltration parameters were then added to each node detailing the infiltration characteristics of the catchment as well as the surface roughness coefficient. XP Storm uses the "Uniform Loss Model" to determine losses due to infiltration through the catchment and a Manning's value of "n" to define catchment roughness.

Links between nodes (catchments) were considered to be natural channels based on 3D contour information obtained from site the survey. For each channel a separate length, slope, shape and value of Manning's "n" surface roughness coefficient was assigned.

To determine the critical storm event, Q20 Storms of varying durations were run through the model with a continuing loss of 5.0mm/hr and an initial loss's assumed from existing site characteristics. From these results the following critical storm durations were obtained at points 'A' and 'B'.

**Table 2: Critical Storm Duration, Existing**

| Discharge Location | Critical Storm (min) |
|---|---|
| Point 'A' | 60 |
| Point 'B' | 60 |

In the absence of any gauged flow data from the catchments, the XP Storm model was calibrated to Rational Method results previously calculated. In order to achieve calibration between the Rational Method calculations and the XP model, no catchment detention characteristics were modelled at this stage.

The 1 in 2, 5, 10, 20, 50 and 100 year ARI storm events of the critical storm duration were run through the model with the same continuing loss as in the critical storm calculation. Values for the initial loss were then refined to minimise the difference between the XP storm model results and rational method results previously calculated. Values for initial loss, Continuing loss and mannings 'n' of 13.5mm/hr, 5mm/hr and 0.08 respectively were determined in the calibrated existing model.

A summary of the Total Catchment Calibration results is as follows:

**Table 3: XP Storm Calibration Results @ Point A:**

| ARI | Rational Method ($m^3/s$) | XP Storm (m3/s) | Difference % |
|---|---|---|---|
| 2 Year | 7.98 | 7.09 | -10% |
| 5 Year | 11.64 | 11.59 | -1% |
| 10 Year | 14.05 | 14.96 | 6% |
| 20 Year | 17.26 | 19.21 | 10% |
| 50 Year | 22.67 | 24.78 | 9% |
| 100 Year | 26.78 | 28.66 | 7% |

**Table 4: XP Storm Calibration Results @ Point B:**

| ARI | Rational Method ($m^3/s$) | XP Storm (m3/s) | Difference % |
|---|---|---|---|
| 2 Year | 1.96 | 1.76 | -10% |
| 5 Year | 2.84 | 2.83 | - |
| 10 Year | 3.42 | 3.65 | 7% |
| 20 Year | 4.19 | 4.63 | 10% |
| 50 Year | 5.49 | 5.79 | 5% |
| 100 Year | 6.48 | 6.70 | 3% |

The model was generally calibrated to within 10% of the rational method calculations for all storm events. This is considered as acceptable and within the bounds of accuracy of the model.

### 5.2.4 Developed Site Model

The existing model was the adjusted to include the developed site characteristics such as an increased impervious area and discharge mitigation measures such as detention basins.

Infiltration data assumed for the existing case was maintained for the undeveloped catchments within the model. Additional infiltration parameters to represent the pervious and impervious nature of the developed catchment were then created and applied to the appropriate catchment areas within the model. Values for Initial loss, continuing loss and Mannings "n" of 2.5mm, 1mm/hr and 0.018 respectively have been assumed to represent the impervious areas of the catchment. Values for Initial loss, continuing loss and Mannings "n" of 12.5mm, 5mm/hr and 0.035 respectively have been assumed to represent the pervious areas of the catchment.

Developed catchments were linked using multi-links allowing the model to perform a 1D analysis of the pipe and overland flow networks.

#### 5.2.4.1 On Site Stormwater Mitigation

On site stormwater mitigation measures will be implemented throughout the future development. Mitigation measures will be implemented to reduce the impact of stormwater discharges and velocities off the site in all storm events. For the purposes of this model the effect of minor mitigation measures such as bio-retention basins and rainwater harvesting tanks will be negligible and therefore only major mitigation measures such as detention basins will be modelled at this stage.

As summary of proposed mitigation measure for this development is as follows:

<u>MINOR MITIGATION MEASURES</u>

- **Bio-Retention Basins**

  Bio-retention basins are designed to capture the Q3mth flows from the catchment for primarily for water quality treatment purposes as described later in this report. It is the intention that these basins be extended 200mm above the permanent detention volume and supplied with small overflow weirs, thus detaining low nuisance flows prior overflowing into the detention basins adjacent.

- **Rainwater Tanks**

  Rainwater tanks will have a negligible impact on peak stormwater runoff, however they will aid in the reduction of stormwater volumetric discharge by capturing water for re-use within the development.

<u>MAJOR MITIGATION MEASURES</u>

- **New Detention Basin**

  It is proposed that two detention basins be constructed within the development adjacent the respective discharge locations 'A' and 'B'. These basins are shown on the attached stormwater management plans in Appendix B of this report. Detention basin volumes and outlet details have been determined using XP storm software and a summary of these characteristics are as follows;

**Table 5: Detention Basin Details**

| | Basin 'A' | Basin 'A' |
|---|---|---|
| **Invert Level (mAHD)** | 47.650 | 32.200 |
| **Secondary Orifice Level (mAHD)** | N/A | 33.750 |
| **Overflow Weir (mAHD)** | N/A | 34.600 |
| **Crest Level (mAHD)** | 50.650 | 35.050 |
| **1 in 20 Year ARI Water Level (mAHD)** | 49.909 | 34.566 |
| **1 in 100 Year ARI Water Level (mAHD)** | 50.472 | 35.008 |
| **Total Storage Volume ($m^3$)** | 5,025 | 10,316 |
| **Low Flow Details** | N/A | 2 x 600mm dia RCP |
| **Secondary Orifice Details** | N/A | 1200 x 1200 grated field inlet. |
| **Outlet Details** | 600mm dia RCP | 1050mm dia. RCP |

### 5.2.5 Developed Site Model Results

Once major mitigation and developed catchment characteristics had been input into the developed model the critical storm duration was again calculated using the same procedure as for the existing model.

**Table 6: Critical Storm Duration, Developed**

| Discharge Location | Critical Storm |
|---|---|
| **A** | 60 |
| **B** | 60 |

The 1 in 2, 5, 10, 20, 50 and 100 year ARI storm events of the critical storm duration were then re-run in the developed model to compare the existing and post development, mitigated peak discharge rates at points 'A', 'B'. Following is a table of calculated results;

**Table 7: Existing vs Developed Discharge Results @ Point A:**

| ARI | Existing ($m^3/s$) | Developed (m3/s) | Difference % |
|---|---|---|---|
| **2 Year** | 7.09 | 7.14 | 1% |
| **5 Year** | 11.59 | 11.26 | -3% |
| **10 Year** | 14.96 | 14.49 | -3% |
| **20 Year** | 19.21 | 18.51 | -4% |
| **50 Year** | 24.78 | 23.92 | -3% |
| **100 Year** | 28.66 | 28.36 | -1% |

**Table 8: Existing vs Developed Discharge Results @ Point B:**

| ARI | Rational Method ($m^3/s$) | XP Storm (m3/s) | Difference % |
|---|---|---|---|
| 2 Year | 1.76 | 1.63 | -7% |
| 5 Year | 2.83 | 2.65 | -6% |
| 10 Year | 3.65 | 3.12 | -14% |
| 20 Year | 4.63 | 3.54 | -24% |
| 50 Year | 5.79 | 4.76 | -18% |
| 100 Year | 6.70 | 5.87 | -12% |

The above results indicate a general decrease in peak flow rates at the existing discharge locations 'A' and 'B'. A slight increase in peak flow rate of 50l/s has been identified in the minor storm at discharge location 'A', however given the magnitude of the flows generated from the catchment this increase is considered negligible.

Water elevation results were also obtained from the existing and developed model, upstream of the existing culverts under the Cunningham Highway. Results were taken at Note N5 (Point A) and Node N2 (Point B). A summary of the results are detailed in the tables below;

**Table 9: U/S Water Levels at 4 x 1650 Culverts Under the Cunningham Highway (Point A)**

| ARI | Existing (mAHD) | Developed (mAHD) | Difference (m) |
|---|---|---|---|
| 2 Year | 45.282 | 45.285 | +0.003 |
| 5 Year | 45.617 | 45.593 | -0.024 |
| 10 Year | 45.850 | 45.816 | -0.034 |
| 20 Year | 46.129 | 46.083 | -0.046 |
| 50 Year | 46.453 | 46.394 | -0.059 |
| 100 Year | 46.898 | 46.700 | -0.198 |

**Table 10: U/S Water Levels at 4 x 1650 Culverts Under the Cunningham Highway (Point B)**

| ARI | Existing (mAHD) | Developed (mAHD) | Difference (m) |
|---|---|---|---|
| 2 Year | 31.370 | 31.334 | -0.036 |
| 5 Year | 31.659 | 31.621 | -0.038 |
| 10 Year | 31.888 | 31.736 | -0.152 |
| 20 Year | 32.269 | 31.858 | -0.411 |
| 50 Year | 32.811 | 32.329 | -0.482 |
| 100 Year | 33.363 | 32.859 | -0.504 |

The above results indicate a general decrease in water levels at discharge locations 'A' and 'B', U/S of the Cunningham Highway. A slight increase in water level of 3mm has been identified in the minor storm at discharge location 'A', however given the magnitude of the flows generated from the catchment this increase is considered negligible.

### 5.2.6 Proposed Infrastructure

The stormwater system will be a conventional gravity network consisting of stormwater and roofwater drainage networks.

#### 5.2.6.1 Stormwater

The stormwater network will be located predominantly within the road network, open space and within drainage easements throughout the development.

Minor drainage infrastructure will consist of conventional pit and pipe drainage. The drainage network will be designed in accordance with values detailed Table 11 below depending on development category.

Major storm flows in excess of the pit and pipe drainage network will be conveyed via natural channels and the road network to points of discharge.

All stormwater outlets into tributaries will be above 1 in 100 year flood levels and discharge velocities will be kept below 0.5m/s & 3m/s for minor & major storms respectively. Where velocities cannot be controlled within these ranges, energy dissipation devices and scour protection will be provided to reduce discharge velocities and the potential for scour.

All outlets will be via concrete headwall and all outlets greater than 375mm diameter will be fitted with trash racks.

#### 5.2.6.2 Roofwater

The roofwater network will consist initially of individual roofwater tanks connected to all residential properties.

Within higher density residential (Lots <600m$^2$), roofwater drainage will be implemented in accordance with Table 11 below. This roofwater network will provide an overflow relief to the rainwater tank system and will be connected into the conventional stormwater network described above.

#### 5.2.6.3 Design Standards

In general the stormwater network for both minor and major storm events will be designed in accordance with;

- Queensland Urban Drainage Design Manual (QUDM) 2007;
- Ipswich City Council Planning Scheme Policy 3, General Works

In addition to the above, the following design and construction principals will apply to this development;

**Table 11: Recommended Design Average Reoccurrence Intervals (QUDM 2007)**

| Development Category | Major (ARI) | Minor (ARI) | Roofwater |
|---|---|---|---|
| High Density Urban Residential | 100 | 10 | QUDM Level (IV) |
| Open Space - Parks | 100 | 1 | N/A |
| Major Road | 100 | 10 | N/A |
| Minor Road | 100 | Relevant to development category | N/A |

## 5.3 STORMWATER QUALITY MANAGEMENT

The existing site is situated within the Deebing Creek Catchment. All references within this report to water quality objectives and design requirements have been based on Ipswich City Council's Planning Scheme Policy, in particular Tables 2.3.1 and 2.3.2.

In recent years there has been a general concern about the impact of development and the quality of stormwater runoff on receiving waters. As a result, local councils (i.e: Ipswich & Brisbane City Councils) have adopted principles of Water Sensitive Urban Design (WSUD), sourced primarily from regional guidelines (i.e: SEQ WSUD Guidelines - Healthy Waterways), to improve discharging stormwater from residential developments.

The use of such WSUD designs has been considered in the master planning and preliminary design phase of stormwater infrastructure for this development. These will be included and detailed further during the Development Application design phase.

For this investigation, reference has been made to the South-East Queensland Healthy Waterways WSUD Technical Guidelines, which is considered in accordance with Ipswich City Council's Best Management Practice, as outlined in their Planning Scheme Policy. In the absence of specific water quality modelling tools, the most applicable and general design criteria, being Brisbane City Council Guidelines for Pollutant Export Modelling in Brisbane – Version 7, have been applied to this Development.

### 5.3.1 Pollutants of Concern

According to the Regional and Council guidelines of reference, the key pollutants generated during the operational phase (post-construction) of development are listed in the table below:

| Pollutant | Subdivisions | Residential Development | Industrial and Commercial Devt # | Service Stations | Carparks |
|---|---|---|---|---|---|
| Litter | yes | yes | yes | yes | yes |
| Sediment | yes | yes | yes | no | yes |
| Oxygen demanding substances (organic and chemical matter) | possibly | no | possibly | no | no |
| Nutrients (N & P) | yes | yes | possibly | possibly | no |
| Pathogens / Faecal coliforms (bacteria & viruses) | yes | yes | possibly | no | no |
| Hydrocarbons (including oil & grease) | yes | possibly | possibly | yes | yes |
| Heavy metals (often associated with fine sediment) | yes | yes | possibly | yes | yes |
| Surfactants (e.g. detergents from car washing) | yes | yes | possibly | possibly | no |
| Organochlorines & organophosphates (e.g. pesticides, herbicides) | unlikely | no | possibly | no | no |
| Thermal pollution (heat) | yes | yes | yes | yes | yes |
| pH altering substances ## | possibly | no | possibly | no | no |

(Source: ICC Planning Scheme Policy)

Based on the above and general engineering best practice, the following key pollutants are recommended to be targeted for treatment:

### 5.3.2 Environmental Values & Water Quality Objetives

Key pollutants to be targeted for trapping and the discharge concentration objectives outlined in ICC (Ref 2) are as follows:

**Table 12: Water Quality Objectives**

| Pollutant | Reduction in the average annual pollutant load discharging from site |
|---|---|
| Suspended Solids (TSS) | 80% |
| Total Phosphorus | 60% |
| Total Nitrogen | 45% |
| Gross Pollutants | 90% |

### 5.3.3 Modelling Assessment/Approach

A quantitative assessment of stormwater runoff quality was considered for the operational phase of the development.

The predicted future (operational phase) discharge concentrations of key pollutants, has been conducted using the *"Model for Urban Stormwater Improvement Conceptualisation (MUSIC)"*. MUSIC is a stormwater quality modelling program that provides estimates of stormwater pollution generation and the performance of stormwater management measures used in series or parallel to form a 'treatment train'. Input parameters and desired Water Quality Objectives (WQOs) specific to Ipswich were adopted in accordance with Council guidelines.

### 5.3.4 Meteorological Data, Source Nodes & Soil Properties

The first step in creating the MUSIC model was to select the appropriate meteorological data set (period and time step) to be used as the basis for the runoff algorithms. Section 2.2 – Meteorological Data of the reference material details the Rainfall Data and Time Step process requirements of the model.

The time step used for the MUSIC modelling process was: Amberley 6 Minutes (1990-99).

The second step taken in creating the MUSIC model was to define 'Source Nodes' or Sub-Catchments. In the absence of data from Ipswich, source nodes for modelling Brisbane Catchments were defined as per the reference material: Guidelines for Pollutant Export Modelling in Brisbane Version 7-Draft for generating pollutant loads and mitigation strategies.

For the purposes of this investigation Urban Residential Source Nodes have been used to model all catchments within this development. An urban residential node is used to describe low to high-density residential areas and activities servicing local neighbourhood needs. It is typical for Urban Residential areas to contain less than 50% total impervious area.

### 5.3.5 Source Nodes

The second step taken in creating the MUSIC model was to define 'Source Nodes' or Sub-Catchments. In the absence of data from Ipswich, source nodes for modelling Brisbane

Catchments were defined as per the reference material: Guidelines for Pollutant Export Modelling in Brisbane Version 7-Draft for generating pollutant loads and mitigation strategies.

For the purposes of this investigation Urban Residential Source Nodes have been used to model all catchments within this development. An urban residential node is used to describe low to high-density residential areas and activities servicing local neighbourhood needs. It is typical for Urban Residential areas to contain less than 50% total impervious area.

**Table 13: Source Node Catchment Details**

| Catchment | Node Type | Area (Ha) | No. Lots | Effective Fraction Impervious |
|---|---|---|---|---|
| Urban Catchment 1 | Urban Residential | 5.845 | 106 | 50% |
| Roofwater 1 | Urban Residential | 2.120 | 106 | 100% |
| Urban Catchment 2a | Urban Residential | 2.680 | 66 | 50% |
| Roofwater 2a | Urban Residential | 1.320 | 106 | 100% |
| Urban Catchment 2b | Urban Residential | 7.516 | 176 | 50% |
| Roofwater 2b | Urban Residential | 3.520 | 106 | 100% |
| Urban Catchment 2c | Urban Residential | 4.011 | 78 | 50% |
| Roofwater 2c | Urban Residential | 1.560 | 106 | 100% |
| | Total | 28.572 | 426 | 65% |

Source Node Parameters for each node type were based on *Brisbane City Council Guidelines for Pollutant Modelling, Version 7 (2003).* The parameters adopted are listed in Appendix C for reference.

### 5.3.6 Treatment Nodes

A range of SQBMPs were considered for inclusion in the development site layout, with information on available SQBMPs primarily sourced from BCC (Ref 3). Each has been reviewed for its suitability for this location. For this particular development the following treatment options have been chosen for incorporation into the development design:

**Figure 1: SQBMP Selection Matrix**

| SQBMP Description | Discussion |
|---|---|
| Rainwater Tanks | Rainwater tanks can be used as an at source treatment device. Rainwater tanks provide removal of gross pollutants and sediments, as it acts in part as a sediment basin. |
| Bio-retention Basins | A bio-retention system is a combination of vegetation and filter substrate that provides treatment of stormwater through filtration, extended detention and some biological uptake. The option to implement bio-retention systems will reduce overland flow, improve pollutant concentrations and provide aesthetic amenity for the site. |

Design details of treatment nodes can be found in Appendix C of this report.

### 5.4 Proposed Treatment Train

From the available SQBMPs, a BMP 'Treatment Train' was developed to target each of the pollutants of concern and was subsequently incorporated into the development site layout. This treatment train is illustrated below.

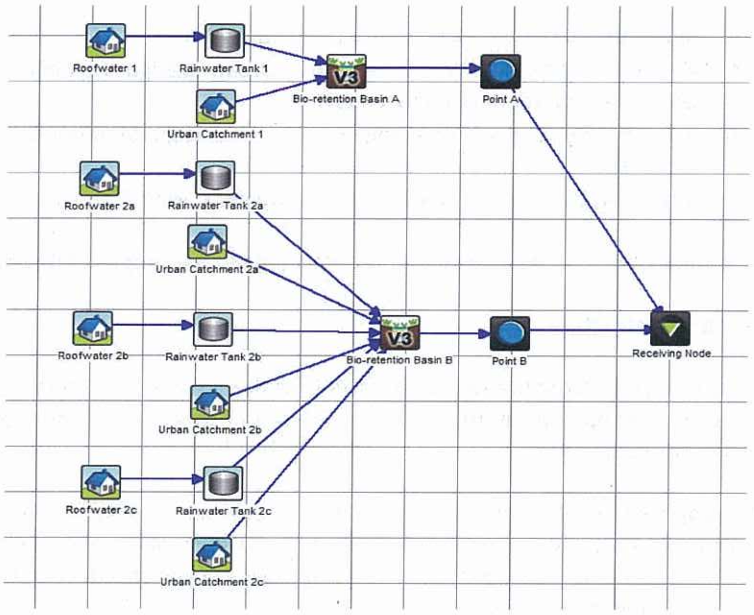


**Figure 2: Treatment Train**

#### 5.4.1 Water Quality Results

The pollutant export levels for the developed phase are summarised below. The estimated pollutant removal efficiencies of each of the SQBMPs are as calculated by the MUSIC model.

**Table 14: Pollutant Export Discharge**

| | TSS (mg/L) | TP (mg/L) | TN (mg/L) | GP (kL/6mins) |
|---|---|---|---|---|
| **Developed Mitigated** | 2.99 | 0.046 | 0.799 | - |
| **Treatment Train Effectiveness** | 94% | 65% | 46% | 100% |
| **ICC WQOs** | 80% | 60% | 45% | 90% |

As shown in Table 14 above, the proposed treatment train reduces the pollutant concentrations by satisfactory amounts. The reduction is compliant with Ipswich City Council's WQOs. It is therefore believed that the proposed development will be acceptable with respect to water quality.

### 5.5 Water Quality Monitoring Program

As this development discharges storm water into Deebing Creek using storm water quality Best Management Practices, no proposal for water quality monitoring has been made. The measures proposed for water quality and water quantity treatment are reasonably well understood and demonstrated in Ipswich. Hence it is anticipated that they are acceptable.

## 6. WATER SUPPLY

### 6.1 INTRODUCTION

In considering the potable water supply system for the development site, the following constraints and guidelines have been taken into account:-

- Ripley Valley Master Planned Area Structure Plan;
- Healthy Waterways Water Sensitive Urban Design Guidelines – Technical Design Guidelines for South East Queensland
- Ipswich City Council Planning Scheme – Design Standards for Water supply and reticulation
- Impact on the Environmental Values of Neighbouring Waterways.

These guidelines have been used as a general guide to the proposed Development.

#### 6.1.1 Existing Infrastructure

The existing infrastructure within the vicinity of the site was predominately constructed as part of recent development on the northern side of Deebing Creek Connection Road, to the south west of the subject site.

The proposed development lies in the water zone serviced by the Ripley Reservoir and it is understood from discussions with Council that sufficient capacity within the reservoir and external reticulation network exists to adequately service this development as well as further development in the vicinity of the subject site.

An existing 600mm dia water main has been constructed within the existing Binnies Road, Road Reserve from the intersection of South Deebing Creek Road/Binnies Road and Deebing Creek Connection Road, east along Binnies Road and continues further east past the future intersection of Binnies Road and Daleys Road.

An existing 450mm water main is constructed within the existing Daleys Road, Road Reserve, extending south as far as the Monterea Road reserve, adjacent the north east boundary of the site. This water main continues east along the Monterea Road Reserve servicing existing and future development. As part of these works the 450mm dia main within Daleys Road will be extended south, to connect with the existing 600mm dia water main at the future intersection of Daleys Road and Binnies Road.

An additional 300mm dia water main has been constructed within the north west corner of the subject site and extends south west along Deeping Creek and north under the Cunningham Highway.

The "Existing Ripley Valley Reservoir" EPANET network was obtained from ICC for the purposes of determining capacity for development in the Ripley Valley. This existing model was used as the base to determine the effects of the development demands on the external networks. Boundary network residual pressures have been derived from this existing model as follows:

**Table 15: External Boundary Conditions**

| Boundary Condition | Residual Head (m) | |
|---|---|---|
| | J34070-450mm dia | J45609-300mm dia |
| Maximum | 102.19 | 102.12 |
| Minimum | 92.34 | 91.44 |

### 6.1.2 Proposed Infrastructure

Initially it is proposed that the development will be serviced from a new 300mm dia main to be extended from the existing 600mm main at the intersection of Daleys Road and Binnies Road. The new 300mm dia main will be extended north to the development site within the Daleys Road, Road Reserve, terminating at the new intersection with Road 1. It is proposed that a single 225mm dia connection will be taken off the new 300mm main within Daleys Road to service the development. A second 225mm dia connection is proposed from the existing 300mm dia water main located within the North West corner the site once the western side of the site is developed.

The new internal 225mm diameter reticulation main will be constructed from the new 300mm dia main within Daleys Road, west along Road 1, terminating at the northern property boundary of the site. This main will service future development to the south of the subject site. A series of 150mm diameter branch mains will be taken off the trunk main to service the individual stages of the development.

The development will be serviced by a series of uPVC pressure reticulation mains between 100mm and 225mm in diameter. These mains will distribute water taken from Ipswich City Council mains water supply at sufficient pressures, rates and volumes to satisfy both Council and the Queensland Fire Service requirements.

The infrastructure layout is such that each stage has a minimum of 2 feeds and at no stage is there to be more than 20 lots on an isolated main as required by Ipswich City Council.

Refer to the attached drawings for further information relating to the location of water infrastructure within the development.

#### 6.1.2.1 Water Demand and Constraints

The water demands for this development which have been used in this investigation have been determined using Ipswich City Council Engineering Works Manual (2004), PSP 3, Part 4-Water Reticulation. The following table sets out these demands:-

**Table 16: System Requirements**

| Item | Criteria | Value (2004) | Value (2007) |
|---|---|---|---|
| 1 | WATER CONSUMPTION (Urban)<br>a) Average Day (AD) to year 2021<br>(i) Residential<br>b) Unaccounted for Water (UFW)<br>c) Mean Day Maximum Month (MDMM)<br>d) Maximum Day (MD)<br>e) Maximum Hour (MH) Residential | 475L/EP/D<br>-<br>1.5AD<br>2.0AD<br>79L/EP/hr | 320L/EP/D<br>50L/EP/D<br>1.5AD<br>2.0AD<br>53.3L/EP/hr |
| 2 | INFRASTRUCTURE REQUIREMENTS<br>a) Reservoir Supply Mains<br>b) Reservoir Storage | MDMM<br>3(MD-MDMM) | MDMM<br>3(MD-MDMM) |
| 3 | RSIDUAL PRESSURE (Urban)<br>a) Minimum allowable pressure<br>b) Minimum pressure in isolated or highly elevated areas<br>c) Max Pressure<br>d) Min Pressure during Fire-Fighting events | 22m (220kPa)<br>16m (160kPa)<br>80m (800kPa)<br>12m (120kPa) | 22m (220kPa)<br>16m (160kPa)<br>80m (800kPa)<br>12m (120kPa) |
| 4 | FIRE FIGHTING DEMAND<br>a) Flows<br>(i) Residential | 15l/s @2hr | 15l/s @2hr |
| 5 | ESTIMATED POPULATION<br>a) Residential | 3.3EP/Lot | 3.3EP/Lot |

Requirements have been based on Ipswich City Council PSP 3, 2004 as they are considered more conservative than 2007

No allowance for future development to the south has been made at this time as it is unclear as to when these lots may come on line and to what extent ne infrastructure will be built in the area when they do.

From this data we have calculated the following Peak Simultaneous Demand (PSD) factors for the development at;

- Residential - 0.073l/s/dwelling;
- Park - 0.05l/s/Ha

### 6.1.2.2 Water Anaylysis

A water network was created and analysed using EPANET 2.0 software and the demand data scheduled earlier in this report. The "Existing Ripley Valley Reservoir" EPANET network was obtained from ICC for the purposes of determining capacity for development in the Ripley

**APPENDIX B**

**STORMWATER QUANTITY**


PLAN
SCALE A

CATCHMENT TABLE

| CATCHMENT | AREA (Ha) |
|---|---|
| N1 | - |
| N2 | 11.27 |
| N3 | 8.23 |
| N4 | - |
| N5 | 3.34 |
| N6 | 1.70 |
| N8 | 17.82 |
| N9 | 2.47 |
| N10 | 2.54 |
| N11 | 17.55 |
| N12 | 17.62 |
| N13 | 2.95 |

PROJECT NORTH

DIMENSIONS IN METRES EXCEPT WHERE SHOWN OTHERWISE. CULVERT AND PIPE SIZES IN MILLIMETRES

A1 UNREDUCED
A3 REDUCED

SCALES
UNREDUCED / REDUCED
A 0 50 100 150 200 250m 1 : 5000 / 1 : 10000

STATUS
COMPLIANCE



| REV. | DATE | AMENDMENT / ISSUE | DWN | DES | CHK | APP |
|---|---|---|---|---|---|---|
| A | 06.10.11 | ISSUE FOR COMPLIANCE A | | | | |

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Phone (07) 3013 4699
Facsimile (07) 3013 4611
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BACKUP AUSTRALIA P/L & P&D HOLDIONGS P/L

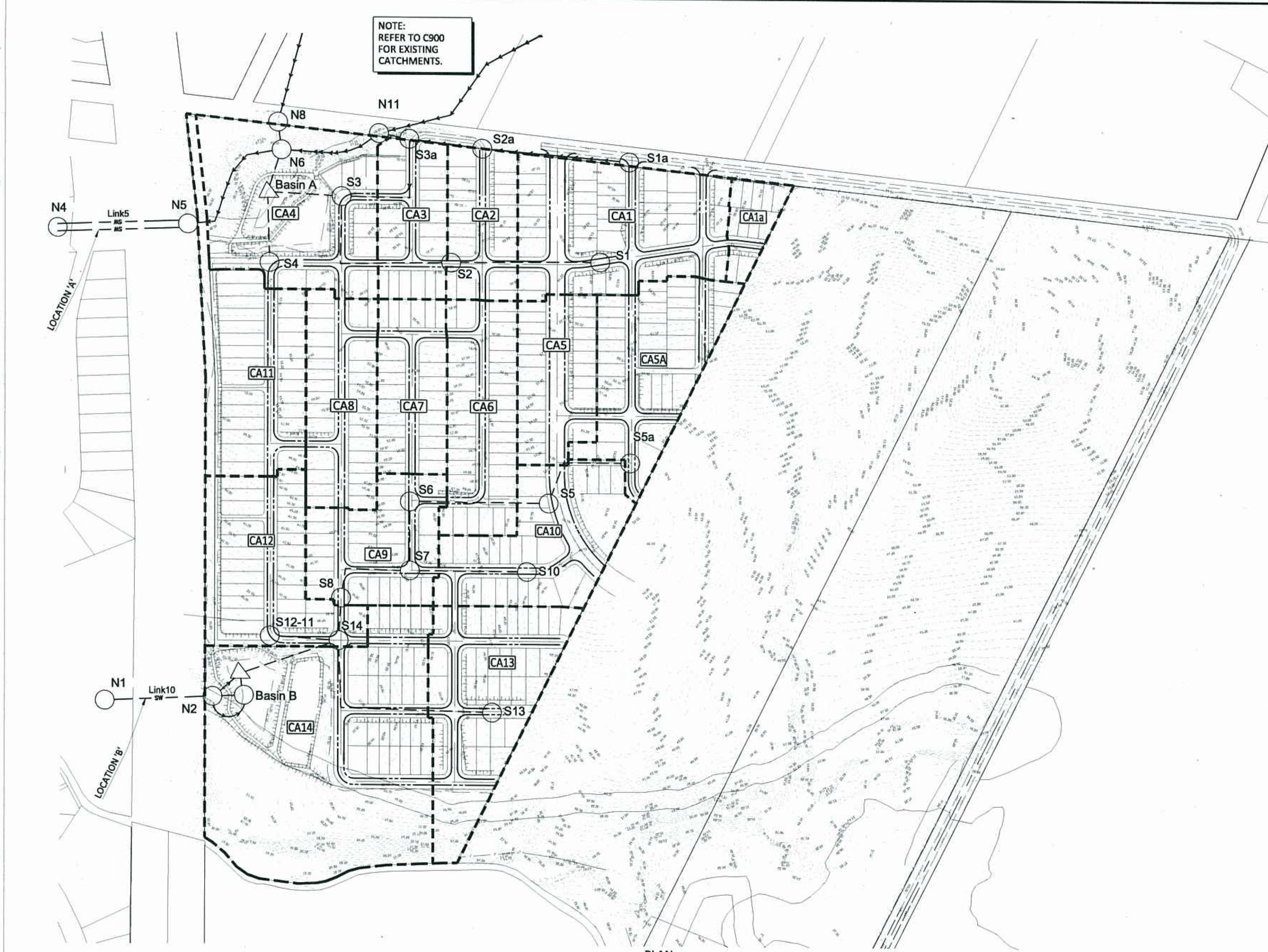
PROJECT
DAYLEYS ROAD RIPLEY

SUBJECT
STORMWATER DRAINAGE CATCHMENT PLAN

PROJECT No.
08199S

DRAWING No. C900
REVISION A

A1




PLAN
SCALE A

## LEGEND

| EXISTING | PROPOSED | |
|---|---|---|
| | | KERB AND CHANNEL |
| | | KERB ONLY |
| | | KERB FLUSH |
| | | KERB INVERT |
| | | EDGE OF BITUMEN |
| | | EXTENT OF BUILDING |
| | | PROPERTY BOUNDARY |
| | | EASEMENT |
| SW | SW | STORMWATER DRAINAGE |
| RW | RW | ROOFWATER DRAINAGE |
| | | OPEN CHANNEL |
| TOP OF BATTER / TOE OF BATTER | TOP OF BATTER / TOE OF BATTER | BATTER |
| TOP OF WALL / TOE OF WALL | TOP OF WALL / TOE OF WALL | RETAINING WALL |
| | | FENCE |
| | | STAGE BOUNDARY |
| | | TREE |
| | | LINE OF VEGETATION |
| | | CATCHMENT BOUNDARY |

## CATCHMENT TABLE

| CATCHMENT | AREA (Ha) |
|---|---|
| CA1 | 2.90 |
| CA1A | 0.47 |
| CA2 | 1.18 |
| CA3 | 1.22 |
| CA4 | 3.07 |
| CA5 | 1.05 |
| CA5A | 2.19 |
| CA6 | 1.06 |
| CA7 | 1.34 |
| CA8 | 1.68 |
| CA9 | 1.67 |
| CA10 | 2.22 |
| CA11 | 2.38 |
| CA12 | 2.23 |
| CA13 | 2.56 |
| CA14 | 5.92 |

PROJECT NORTH

DIMENSIONS IN METRES EXCEPT WHERE SHOWN OTHERWISE. CULVERT AND PIPE SIZES IN MILLIMETRES

A1 UNREDUCED
A3 REDUCED

SCALES
UNREDUCED / REDUCED
A 0 20 40 60 80 100m 1 : 2000 / 1 : 4000

STATUS
# COMPLIANCE



| REV. | DATE | AMENDMENT / ISSUE | DWN | DES | CHK | APP |
|---|---|---|---|---|---|---|
| A | 06.10.11 | ISSUE FOR COMPLIANCE A | | | | |

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Brisbane Australia
Phone (07) 3013 4699
Facsimile (07) 3013 4611
mail@bornhorstward.com.au
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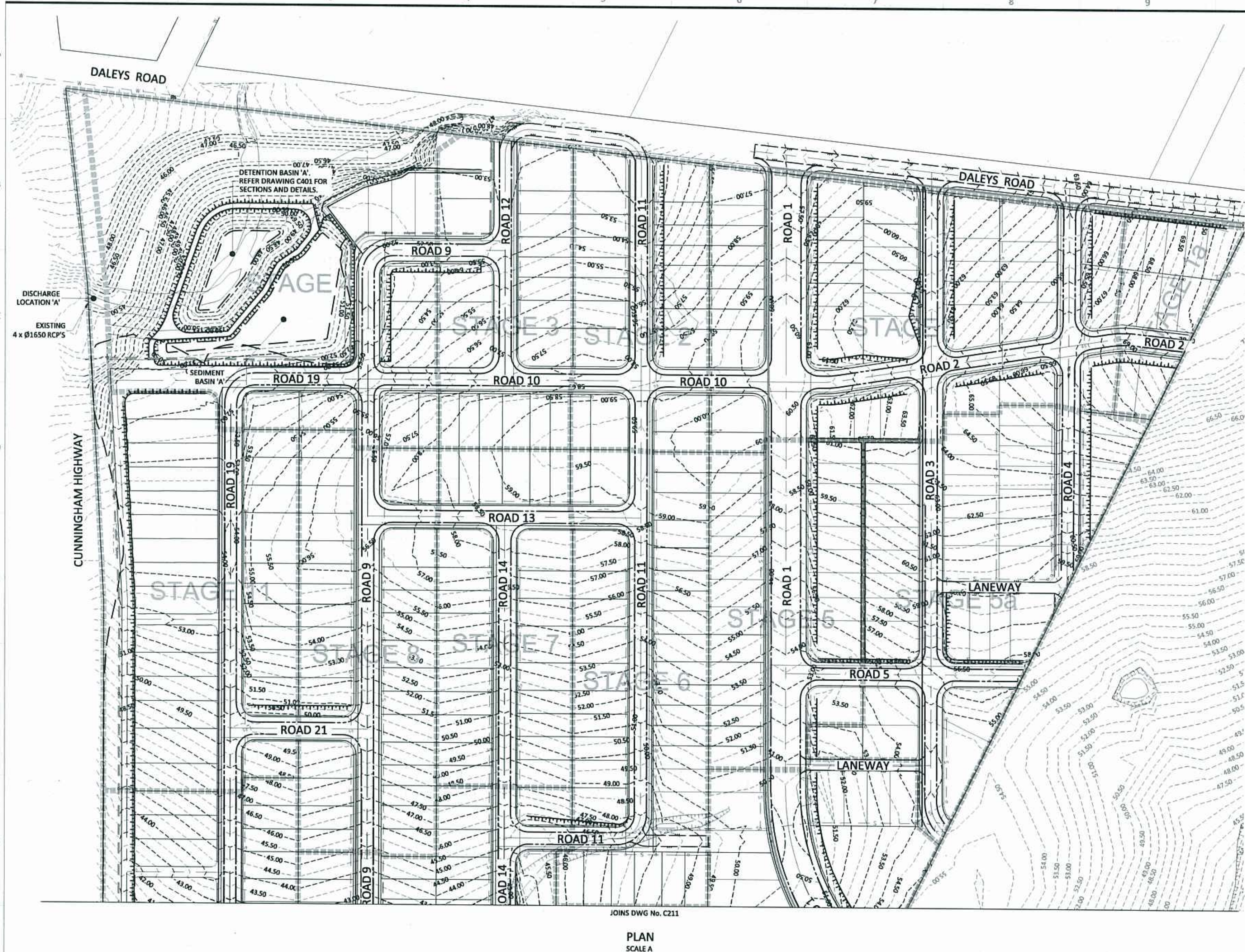
PROJECT
DAYLEYS ROAD RIPLEY

SUBJECT
STORMWATER DRAINAGE
CATCHMENT PLAN
DEVELOPED SCENARIO

PROJECT No.
08199S

DRAWING No. C901
REVISION A




PLAN
SCALE A

## LEGEND

| EXISTING | PROPOSED | |
|---|---|---|
| | | KERB AND CHANNEL |
| | | KERB ONLY |
| | | KERB FLUSH |
| | | EDGE OF BITUMEN |
| | | EDGE OF SHOULDER |
| | | CONTROL LINE |
| | | PROPERTY BOUNDARY |
| | | EASEMENT |
| SW | SW | STORMWATER DRAINAGE |
| | | OPEN CHANNEL |
| | | STONE PITCHING |
| TOP OF BATTER / TOE OF BATTER | TOP OF BATTER / TOE OF BATTER | BATTER |
| TOP OF WALL / TOE OF WALL | TOP OF WALL / TOE OF WALL | RETAINING WALL |
| 52 | 52 | CONTOURS |
| | | FENCE |
| S | S | SEWER |
| RM | RM | SEWER RISING MAIN |
| W | W | WATER |
| | | FOOTPATH |
| | | PRAM RAMP |
| | | STAGE BOUNDARY |
| | | TREE |
| | | LINE OF VEGETATION |

PROJECT NORTH

DIMENSIONS IN METRES EXCEPT WHERE SHOWN OTHERWISE. CULVERT AND PIPE SIZES IN MILLIMETRES

A1 UNREDUCED
A3 REDUCED

SCALES
A 0 10 20 30 40 50m
UNREDUCED / REDUCED 1 : 1000 / 1 : 2000

STATUS
COMPLIANCE



| REV. | DATE | AMENDMENT / ISSUE | DWN | DES | CHK | APP |
|---|---|---|---|---|---|---|
| A | 06.10.11 | ISSUE FOR COMPLIANCE A | | | | |

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Brisbane Australia
Phone (07) 3013 4699
Facsimile (07) 3013 4611
mail@bornhorstward.com.au
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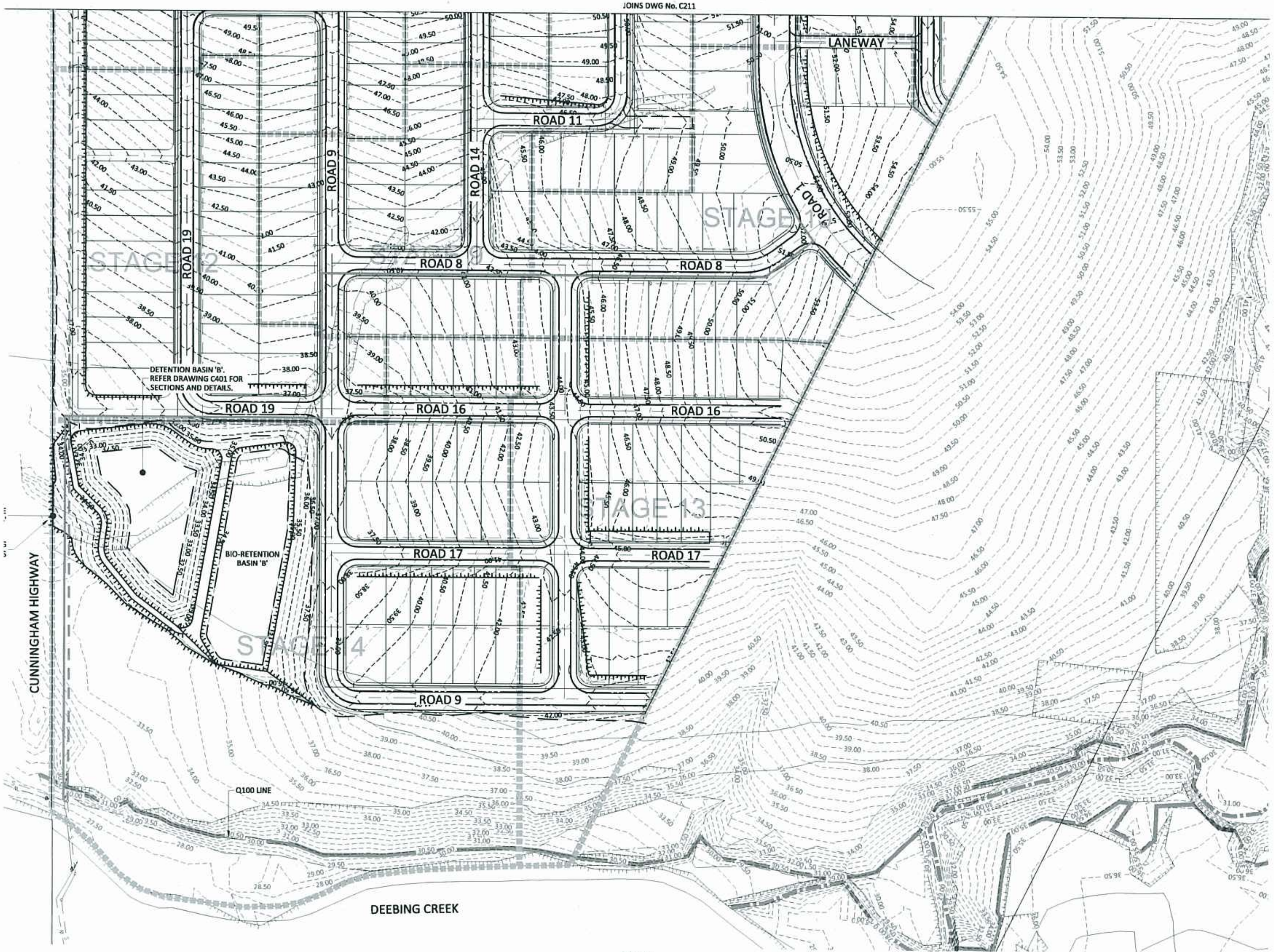
PROJECT
DAYLEYS ROAD RIPLEY

SUBJECT
ROADWORKS AND DRAINAGE LAYOUT PLAN SHEET 1

PROJECT No. 08199S
DRAWING No. C310
REVISION A

A1

JOINS DWG No. C211




PLAN
SCALE A

## LEGEND

| EXISTING | PROPOSED | |
|---|---|---|
| | | KERB AND CHANNEL |
| | | KERB ONLY |
| | | KERB FLUSH |
| | | EDGE OF BITUMEN |
| | | EDGE OF SHOULDER |
| | | CONTROL LINE |
| | | PROPERTY BOUNDARY |
| | | EASEMENT |
| SW | SW | STORMWATER DRAINAGE |
| | | OPEN CHANNEL |
| | | STONE PITCHING |
| TOP OF BATTER / TOE OF BATTER | TOP OF BATTER / TOE OF BATTER | BATTER |
| TOP OF WALL / TOE OF WALL | TOP OF WALL / TOE OF WALL | RETAINING WALL |
| 52 | 52 | CONTOURS |
| | | FENCE |
| S | S | SEWER |
| RM | RM | SEWER RISING MAIN |
| W | W | WATER |
| | | FOOTPATH |
| | | PRAM RAMP |
| | | STAGE BOUNDARY |
| | | TREE |
| | | LINE OF VEGETATION |

PROJECT NORTH

DIMENSIONS IN METRES EXCEPT WHERE SHOWN OTHERWISE. CULVERT AND PIPE SIZES IN MILLIMETRES

A1 UNREDUCED
A3 REDUCED

SCALES
UNREDUCED / REDUCED
A 0 10 20 30 40 50m 1 : 1000 / 1 : 2000

STATUS
# COMPLIANCE



| REV. | DATE | AMENDMENT / ISSUE | DWN | DES | CHK | APP |
|---|---|---|---|---|---|---|
| A | 06.10.11 | ISSUE FOR COMPLIANCE A | | | | |

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Brisbane Australia
Phone (07) 3013 4699
Facsimile (07) 3013 4611
mail@bornhorstward.com.au
www.bornhorstward.com.au

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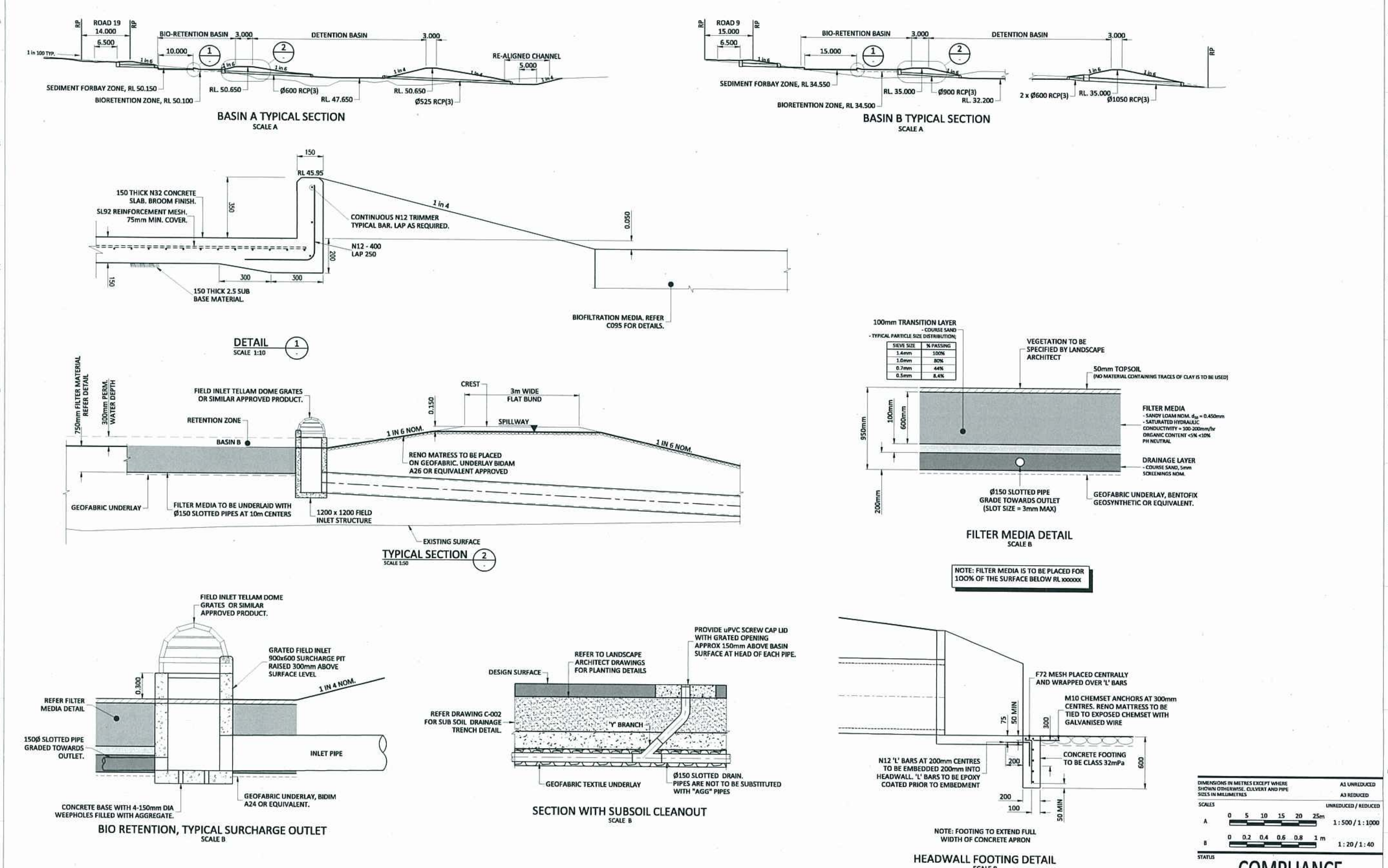
PROJECT
DAYLEYS ROAD RIPLEY

SUBJECT
ROADWORKS AND DRAINAGE LAYOUT PLAN SHEET 2

PROJECT No.
08199S

DRAWING No.
C311

REVISION
A




STATUS

# COMPLIANCE



| REV. | DATE | AMENDMENT / ISSUE | DWN | DES | CHK | APP |
|---|---|---|---|---|---|---|
| A | 06.10.11 | ISSUE FOR COMPLIANCE A | | | | |

ASSOCIATED CONSULTANTS | APPROVED | CHECKED
RPEQ
DATE | DATE

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Bornhorst & Ward Pty Ltd
Brisbane Australia
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Facsimile (07) 3013 4611
mail@bornhorstward.com.au
www.bornhorstward.com.au

CLIENT: BACKUP AUSTRALIA P/L & P&D HOLDIONGS P/L

PROJECT: DAYLEYS ROAD RIPLEY

SUBJECT: TYPICAL BASIN CROSS SECTIONS

PROJECT No. 08199S

DRAWING No. C401

REVISION A

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# INITIAL PEAK FLOW CALCULATIONS FOR RURAL CATCHMENTS

**Based on QUDM 4.00 'Catchment Hydrology'**

Written by BB 14/08/09

Job Number: 08199_S
Job Name: Daleys Road
Cathment A - 60min Critical Storm

IFD : Ipswich

Designed: BB
Checked:

**TOTAL RUNOFF COEFF.**

| No. | Area (Ha) | C10 | Total C10 |
|---|---|---|---|
| N10 | 2.5 | 0.53 | |
| N9 | 24.756 | 0.53 | |
| N8 | 17.782 | 0.53 | |
| N7 | 1.707 | 0.53 | |
| N6 | 5.499 | 0.53 | |
| N5 | 3.349 | 0.53 | |
| N13 | 2.987 | 0.53 | |
| N12 | 17.586 | 0.53 | |
| N11 | 17.527 | 0.53 | |
| 10 | | | |
| | 93.693 | | 0.53 |

**EXISTING TIME OF CONCENTRATION**

| | | |
|---|---|---|
| Bransby-Williams Equation | Channel Slope (%) | 4.8 |
| | Flow Length (km) | 1.5 |
| | Area | 93.693 |
| | Travel Time (min) | 29.50 |
| Modified Friends Equation | Channel Slope (%) | 4.8 |
| | Flow Length (km) | 1.5 |
| | Area | 93.693 |
| | Left Bank Slope (1 in __) | 10 |
| | Right Bank Slope (1 in __) | 10 |
| | Base width (m) | 0.5 |
| | Assumed Flood Depth (m) | 0.5 |
| | Stream Slope (1=uniform, 0=variable) | 1 |
| | Mannings 'n' | 0.055 |
| | Wetted Perimeter (m) | 10.55 |
| | Channel area (m2) | 2.75 |
| | Travel Time | 29.38 |
| | Total Time (Tc) | 29.4 |

**PEAK RUNOFF CALCULATIONS**

| ARI Storm (yr) | Rainfall Intenciety (mm/hr) | Coeficient of Runoff | Peak Runoff Rates (m^3/s) |
|---|---|---|---|
| 1 | 52 | 0.42 | 5.77 |
| 2 | 68 | 0.45 | 7.98 |
| 5 | 89 | 0.50 | 11.64 |
| 10 | 102 | 0.53 | 14.05 |
| 20 | 119 | 0.56 | 17.26 |
| 50 | 143 | 0.61 | 22.67 |
| 100 | 162 | 0.64 | 26.78 |

**PEAK FLOWS FOR FREQUENT EVENTS**

| ARI Storm (yr) | Percentage of Q1 (%) | Peak Runoff Rates (m^3/s) |
|---|---|---|
| 1mth | 25% | 1.44 |
| 2mth | 40% | 2.31 |
| 3mth | 50% | 2.89 |
| 4mth | 60% | 3.46 |
| 6mth | 75% | 4.33 |
| 9mth | 90% | 5.19 |
| 12mth | 100% | 5.77 |

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# INITIAL PEAK FLOW CALCULATIONS FOR RURAL CATCHMENTS

**Based on QUDM 4.00 'Catchment Hydrology'**

Written by BB 14/08/09

Job Number: **08199_S**
Job Name: **Daleys Road**
**Cathment B - 90min Critical Storm**

IFD : Ipswich

Designed: BB
Checked:

**TOTAL RUNOFF COEFF.**

| No. | Area (Ha) | C10 | Total C10 |
|---|---|---|---|
| N2 | 8.231 | 0.53 | |
| N3 | 11.273 | 0.53 | |
| | 0 | 0.00 | |
| | 0 | 0.00 | |
| | 0 | 0.00 | |
| | 0 | 0.00 | |
| | 0 | 0.00 | |
| | 0 | 0.00 | |
| | 0 | 0.00 | |
| | **19.504** | | **0.53** |

**EXISTING TIME OF CONCENTRATION**

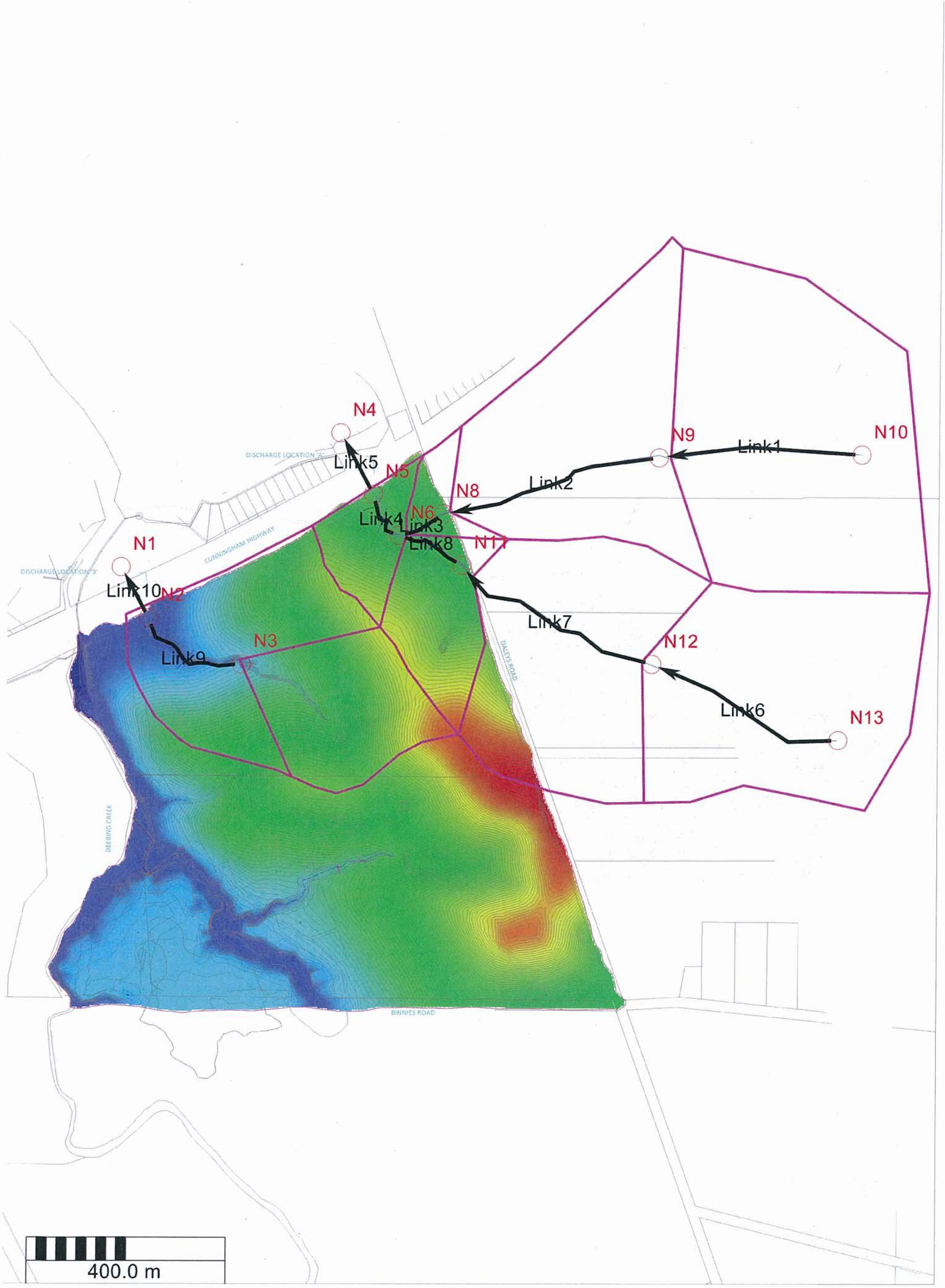
| | | |
|---|---|---|
| Bransby-Williams Equation | Channel Slope (%) | 6 |
| | Flow Length (km) | 1 |
| | Area | 19.504 |
| | Travel Time (min) | 21.05 |
| Modified Friends Equation | Channel Slope (%) | 6 |
| | Flow Length (km) | 1 |
| | Area | 19.504 |
| | Left Bank Slope (1 in __) | 10 |
| | Right Bank Slope (1 in __) | 10 |
| | Base width (m) | 0.5 |
| | Assumed Flood Depth (m) | 0.5 |
| | Stream Slope (1=uniform, 0=variable) | 1 |
| | Mannings 'n' | 0.055 |
| | Wetted Perimeter (m) | 10.55 |
| | Channel area (m2) | 2.75 |
| | Travel Time | 20.96 |
| | **Total Time (Tc)** | **21.0** |

**PEAK RUNOFF CALCULATIONS**

| ARI Storm (yr) | Rainfall Intenciety (mm/hr) | Coeficient of Runoff | Peak Runoff Rates (m^3/s) |
|---|---|---|---|
| 1 | 62 | 0.42 | **1.42** |
| 2 | 80 | 0.45 | **1.96** |
| 5 | 104 | 0.50 | **2.84** |
| 10 | 119 | 0.53 | **3.42** |
| 20 | 139 | 0.56 | **4.19** |
| 50 | 166 | 0.61 | **5.49** |
| 100 | 188 | 0.64 | **6.48** |

**PEAK FLOWS FOR FREQUENT EVENTS**

| ARI Storm (yr) | Percentage of Q1 (%) | Peak Runoff Rates (m^3/s) |
|---|---|---|
| 1mth | 25% | **0.35** |
| 2mth | 40% | **0.57** |
| 3mth | 50% | **0.71** |
| 4mth | 60% | **0.85** |
| 6mth | 75% | **1.06** |
| 9mth | 90% | **1.28** |
| 12mth | 100% | **1.42** |

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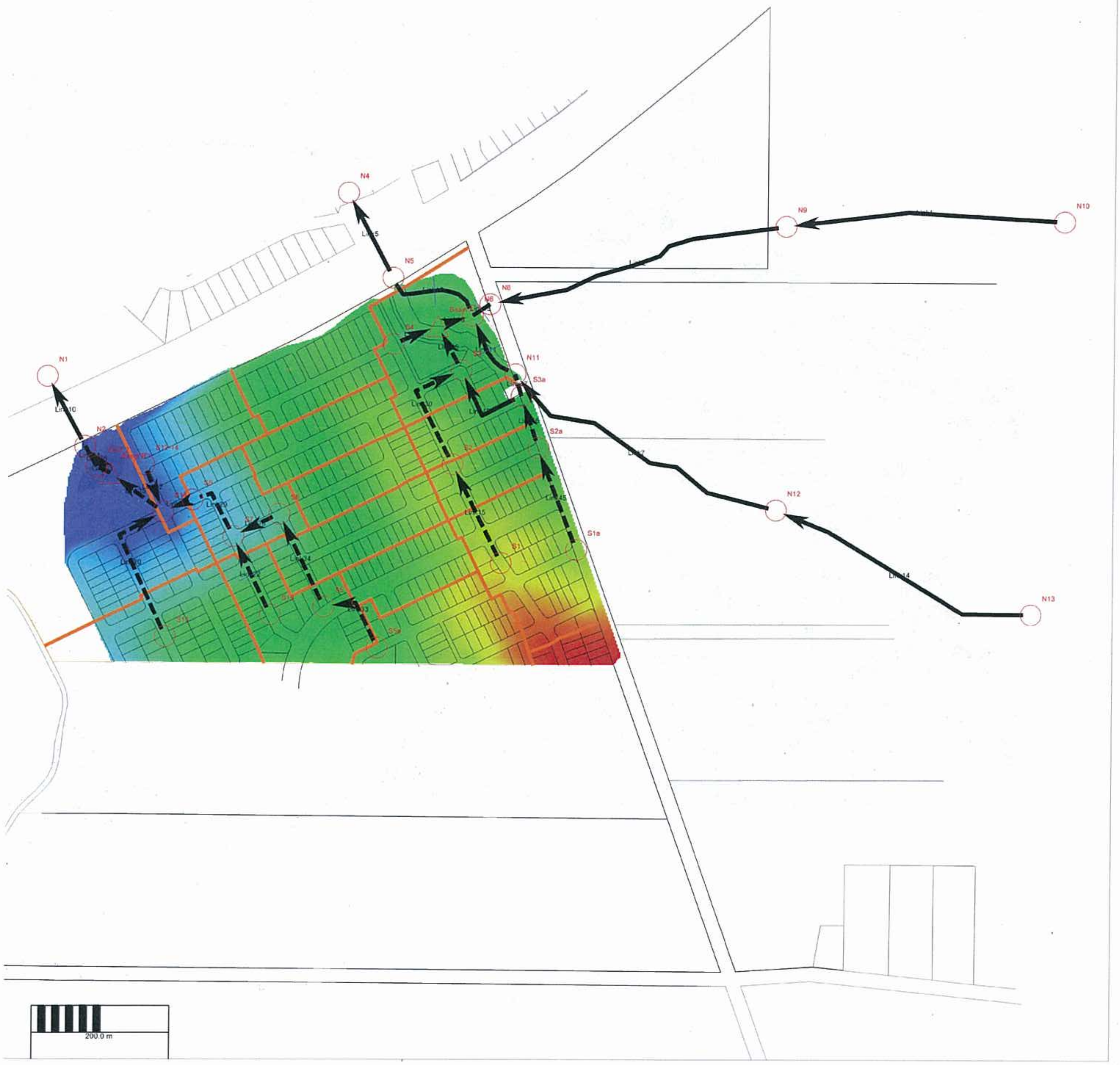


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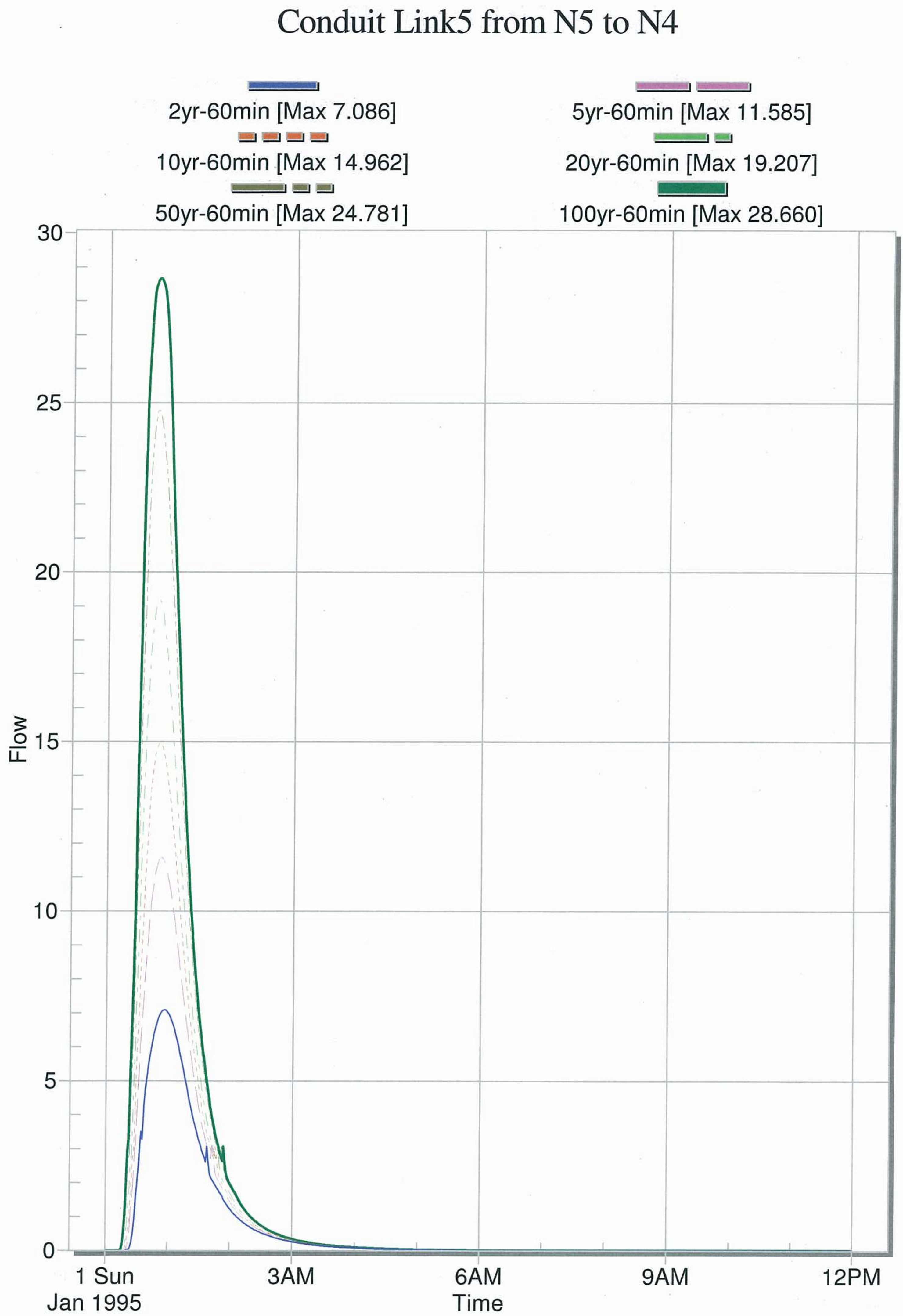
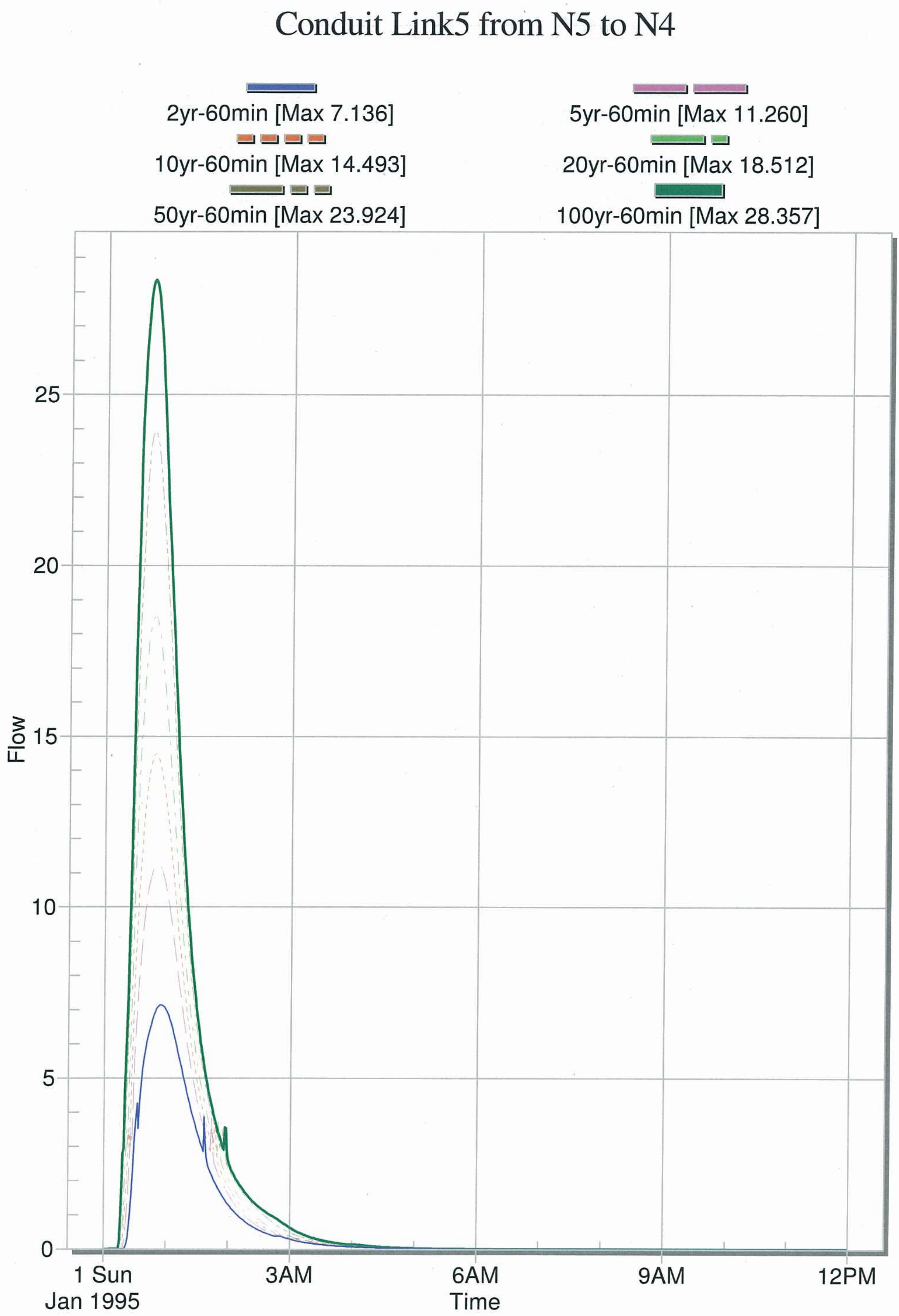
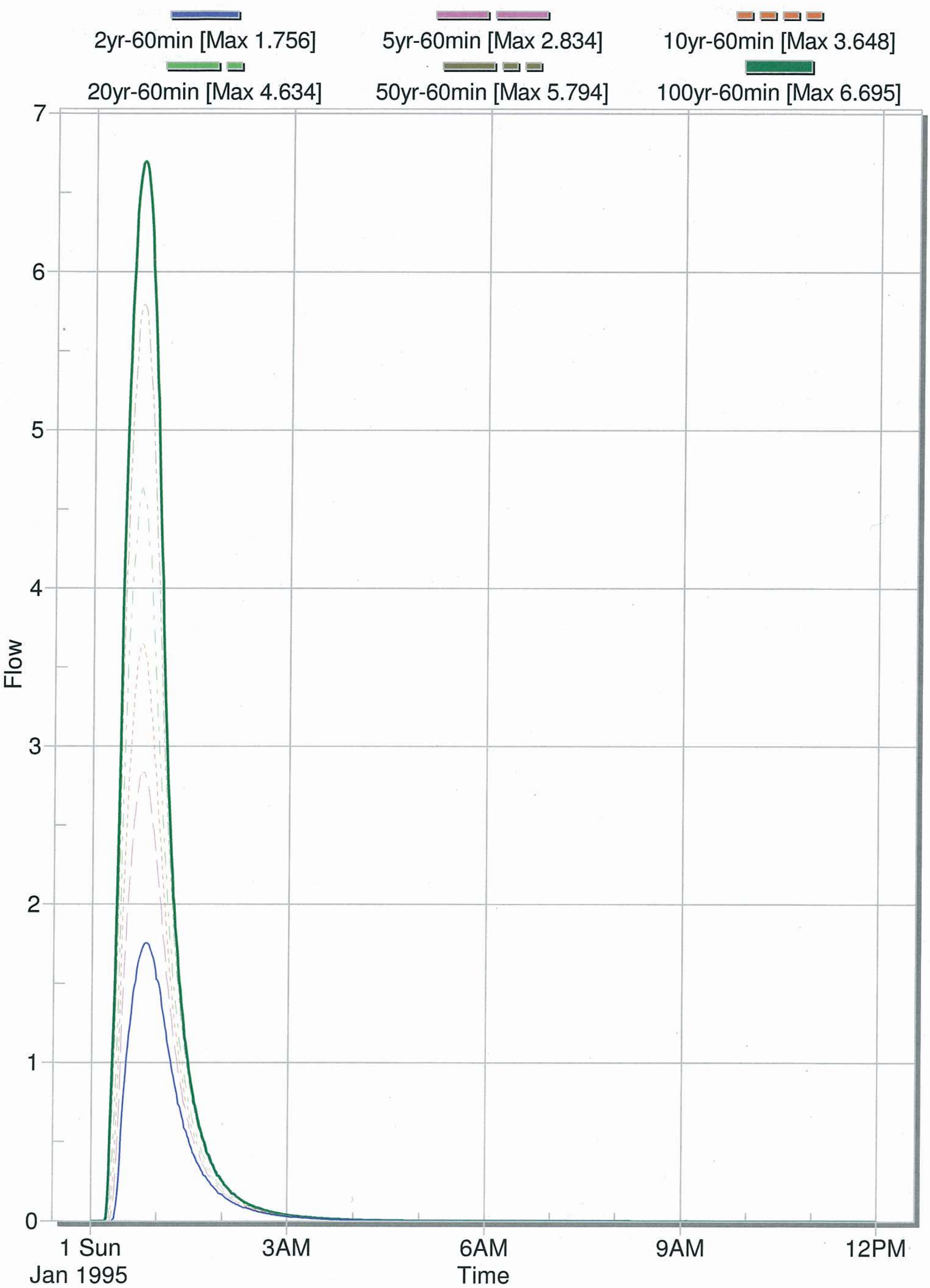


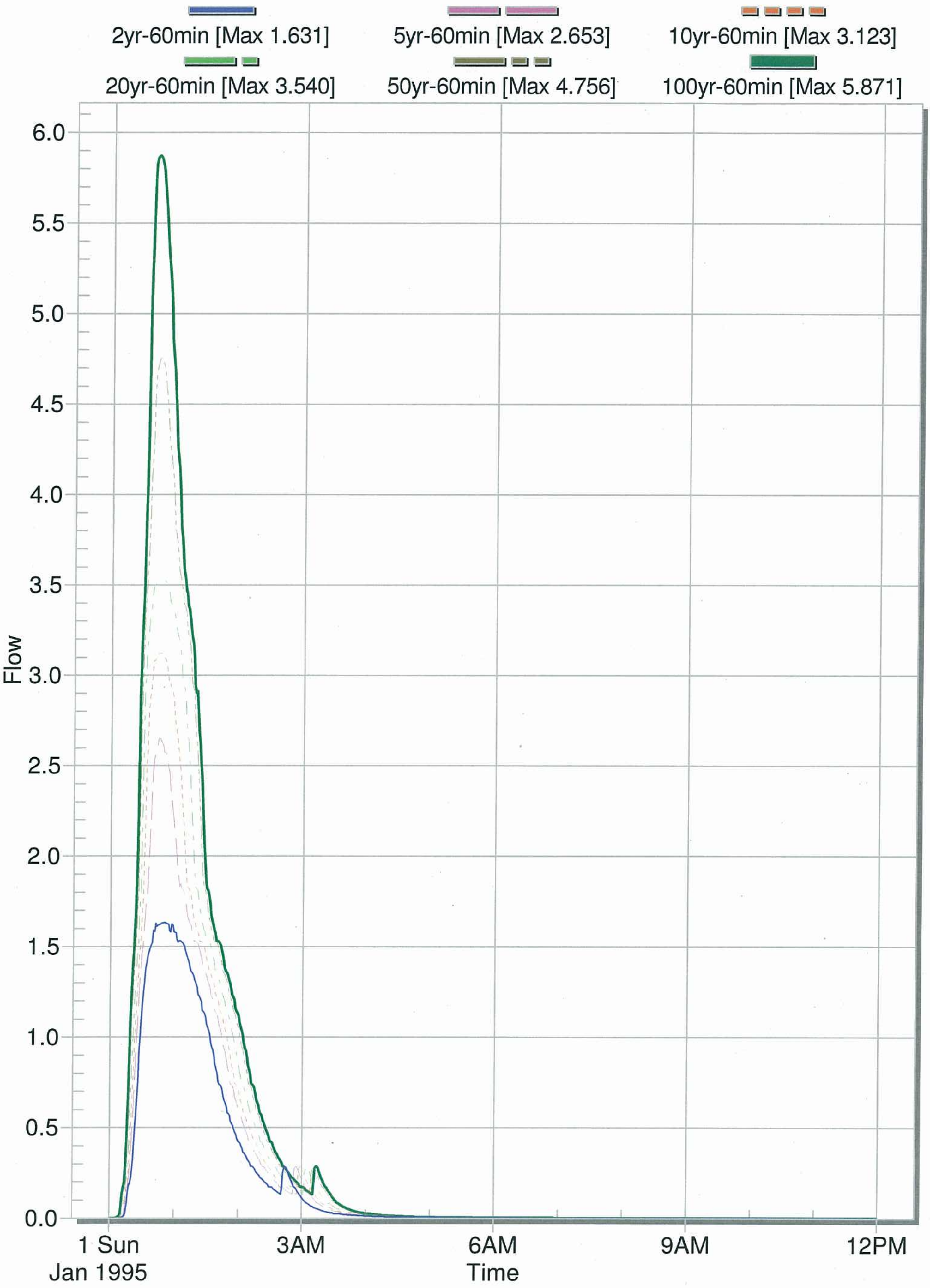
Conduit Link5 from N5 to N4
2yr-60min [Max 7.086]
5yr-60min [Max 11.585]
10yr-60min [Max 14.962]
20yr-60min [Max 19.207]
50yr-60min [Max 24.781]
100yr-60min [Max 28.660]
30
25
20
15
10
5
0
Flow
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time

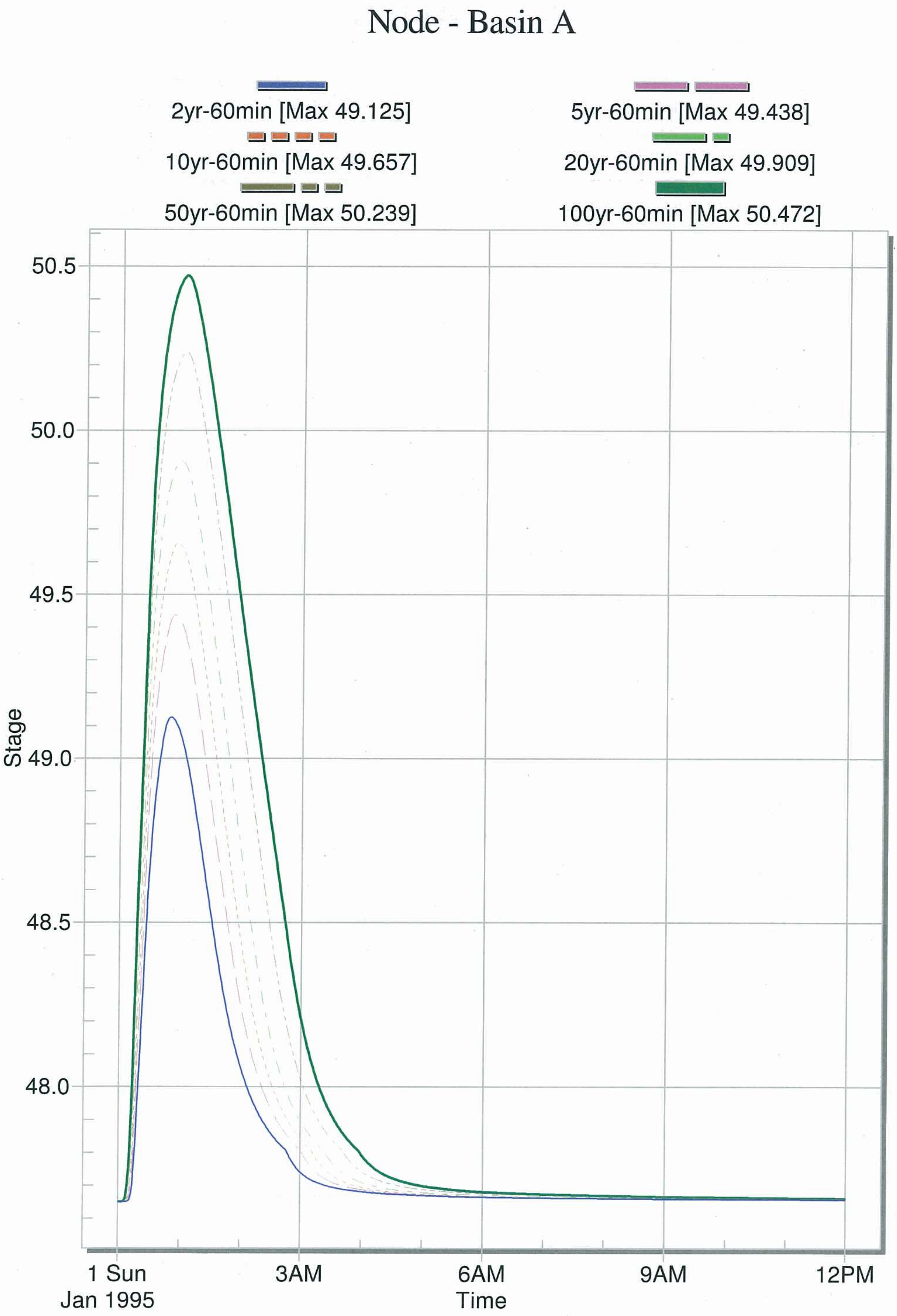

Conduit Link5 from N5 to N4
2yr-60min [Max 7.136]
5yr-60min [Max 11.260]
10yr-60min [Max 14.493]
20yr-60min [Max 18.512]
50yr-60min [Max 23.924]
100yr-60min [Max 28.357]
Flow
25
20
15
10
5
0
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time

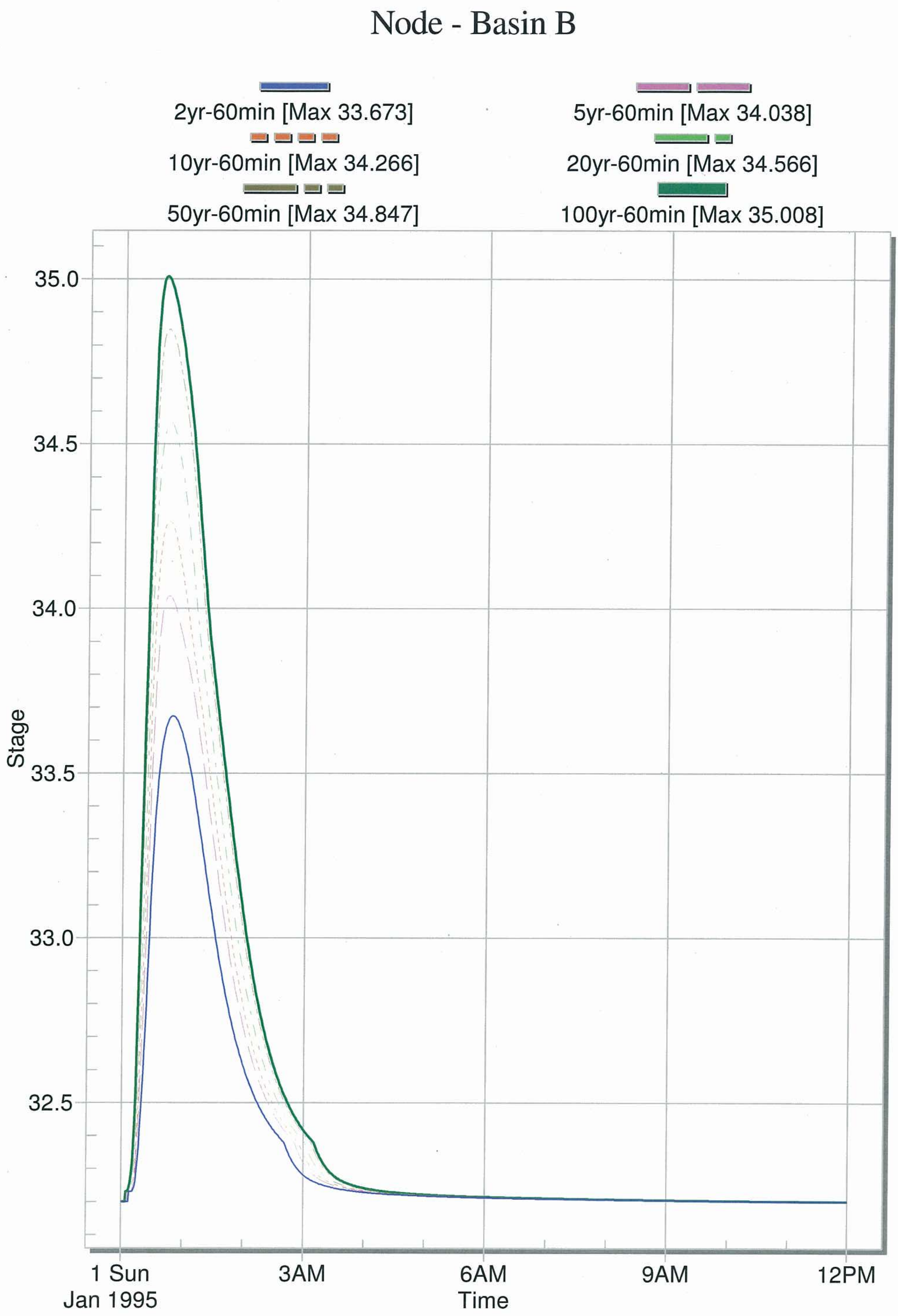
# Conduit Link10 from N2 to N1

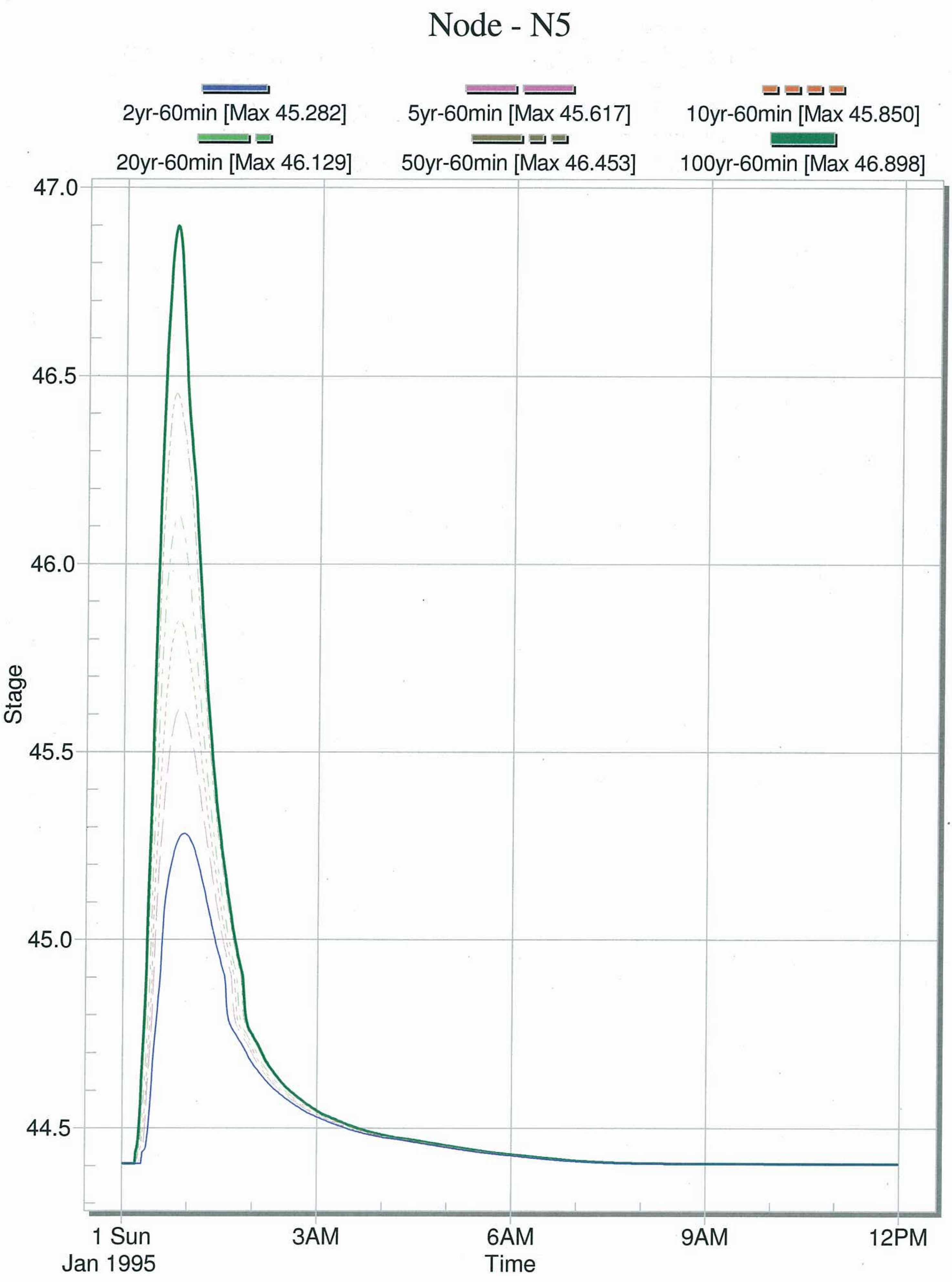


# Conduit Link10 from N2 to N1



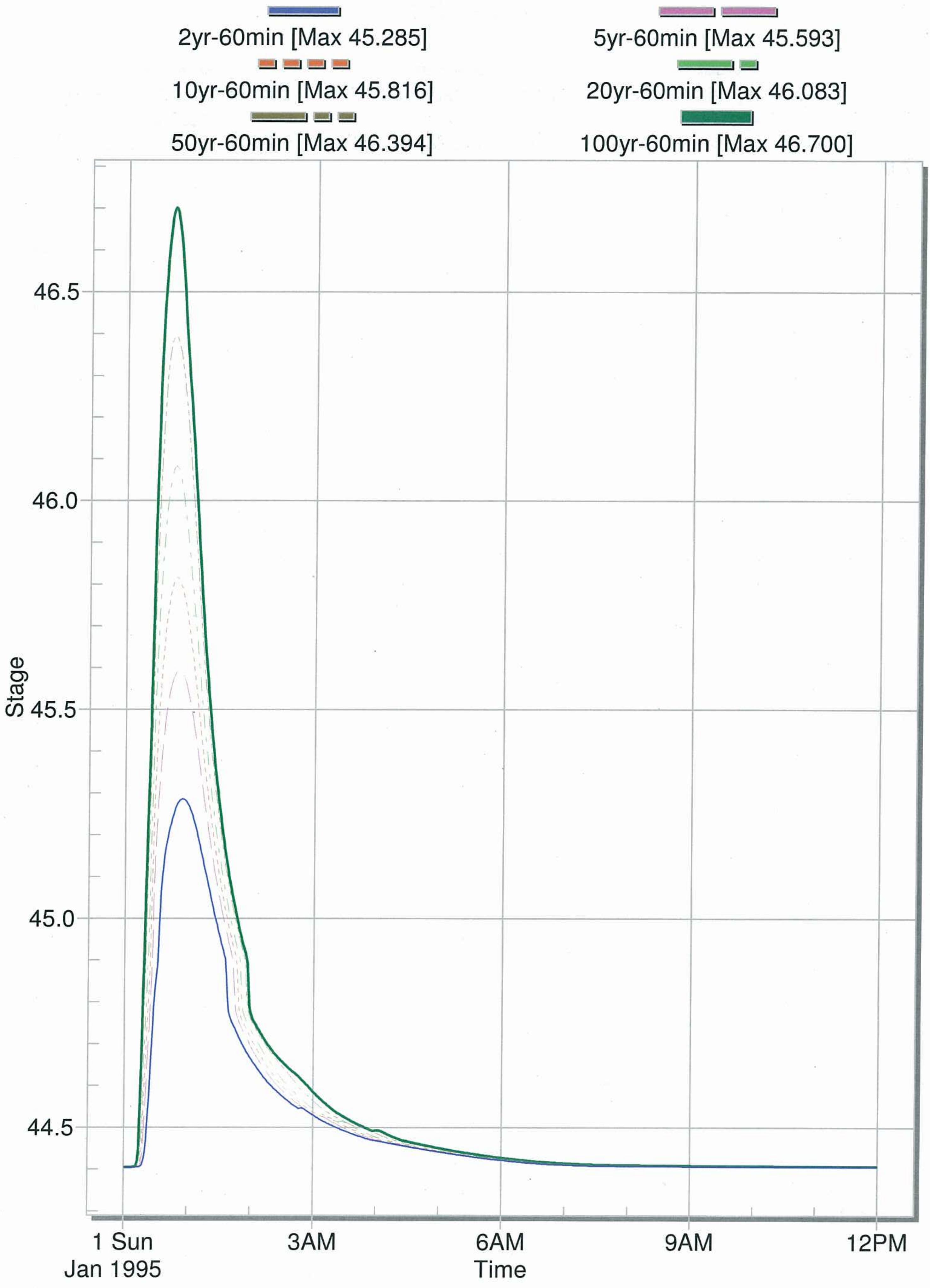

Node - Basin A
2yr-60min [Max 49.125]
5yr-60min [Max 49.438]
10yr-60min [Max 49.657]
20yr-60min [Max 49.909]
50yr-60min [Max 50.239]
100yr-60min [Max 50.472]
50.5
50.0
49.5
49.0
48.5
48.0
Stage
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time

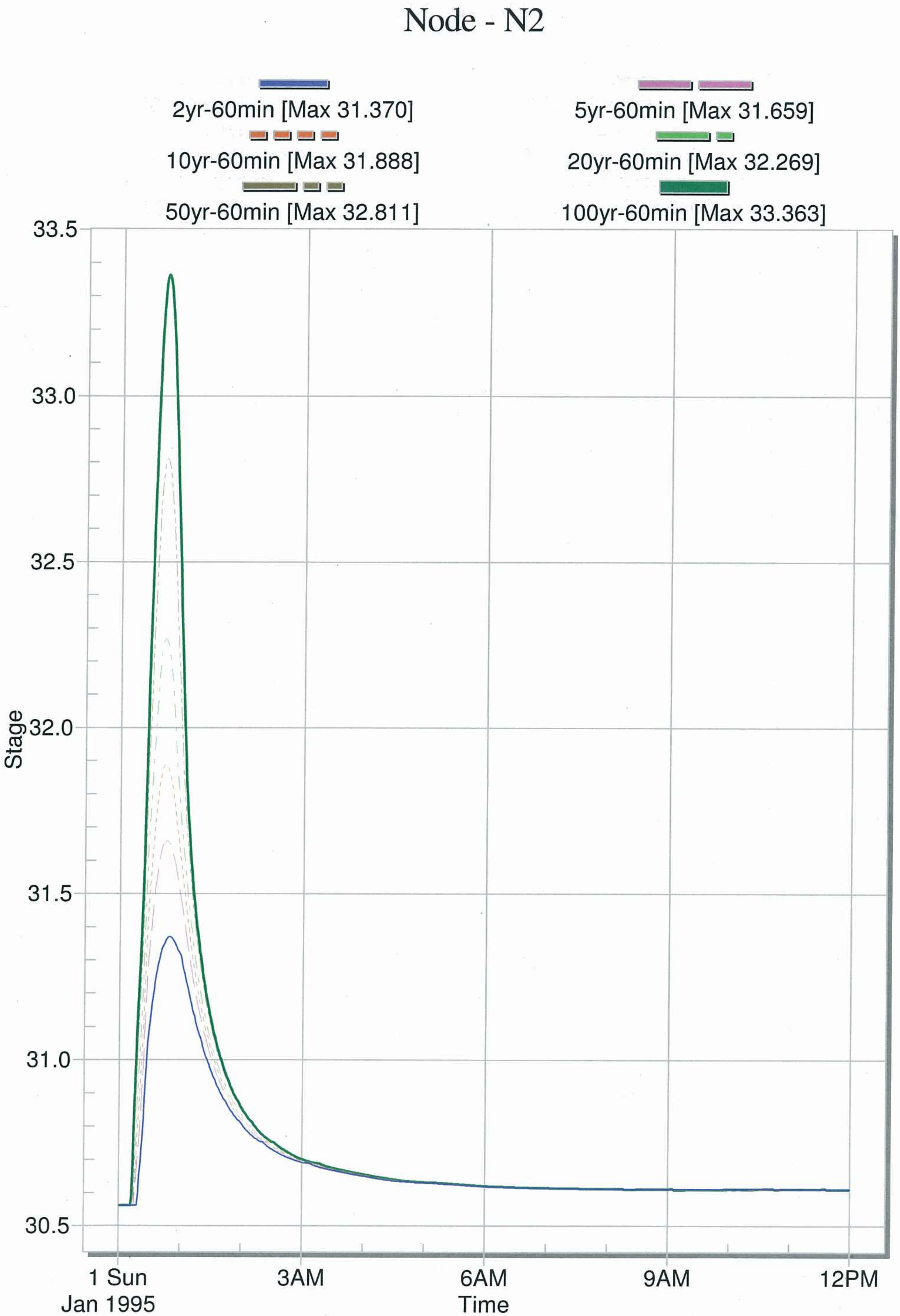

Node - Basin B
2yr-60min [Max 33.673]
5yr-60min [Max 34.038]
10yr-60min [Max 34.266]
20yr-60min [Max 34.566]
50yr-60min [Max 34.847]
100yr-60min [Max 35.008]
Stage
35.0
34.5
34.0
33.5
33.0
32.5
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time

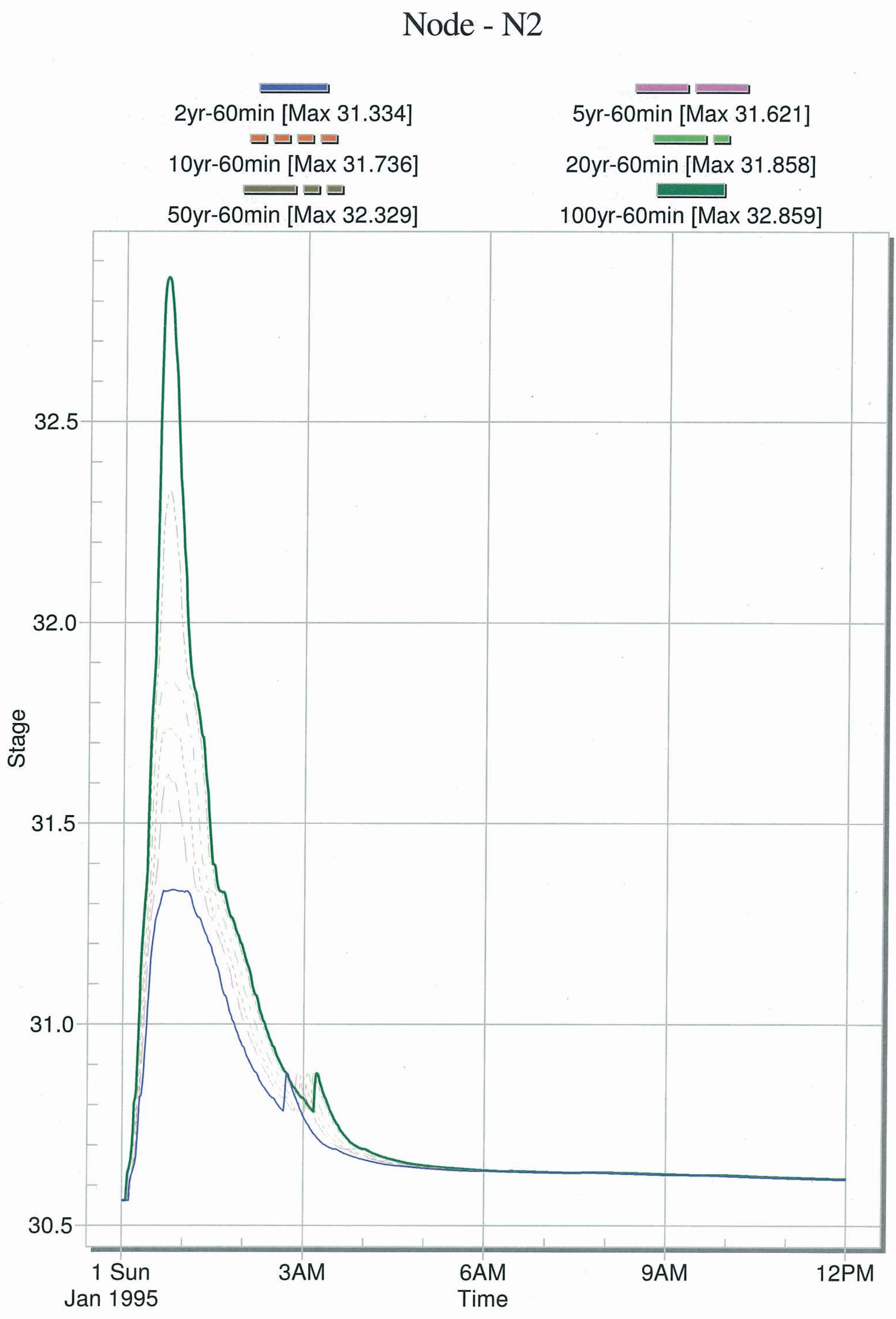


Node - N5
2yr-60min [Max 45.282]
5yr-60min [Max 45.617]
10yr-60min [Max 45.850]
20yr-60min [Max 46.129]
50yr-60min [Max 46.453]
100yr-60min [Max 46.898]
47.0
46.5
46.0
45.5
45.0
44.5
Stage
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time

# Node - N5


2yr-60min [Max 45.285]
5yr-60min [Max 45.593]
10yr-60min [Max 45.816]
20yr-60min [Max 46.083]
50yr-60min [Max 46.394]
100yr-60min [Max 46.700]
46.5
46.0
45.5
45.0
44.5
Stage
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time


Node - N2
2yr-60min [Max 31.370]
5yr-60min [Max 31.659]
10yr-60min [Max 31.888]
20yr-60min [Max 32.269]
50yr-60min [Max 32.811]
100yr-60min [Max 33.363]
Stage
33.5
33.0
32.5
32.0
31.5
31.0
30.5
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time



Node - N2
2yr-60min [Max 31.334]
5yr-60min [Max 31.621]
10yr-60min [Max 31.736]
20yr-60min [Max 31.858]
50yr-60min [Max 32.329]
100yr-60min [Max 32.859]
Stage
32.5
32.0
31.5
31.0
30.5
1 Sun
Jan 1995
3AM
6AM
9AM
12PM
Time

APPENDIX C

STORMWATER QUALITY

**Table 19: Pollutants Typically Generated During the Construction Phase (ICC, Ref 2)**

| Pollutant | Source |
|---|---|
| Litter | Paper, construction packaging, food packaging, cement bags, off-cuts |
| Sediment | Unprotected exposed soils and stockpiles during earthworks and building |
| Hydrocarbons | Fuel and oil spills, leaks from construction equipment |
| Toxic materials | Cement slurry, asphalt prime, solvents, cleaning agents, washwaters (eg. from tile works) |
| pH altering substances | Acid sulphate soils, cement slurry and washwaters |

**Table 20: Recommended MUSIC Runoff Generation Parameters**

| Parameter | Urban Residential |
|---|---|
| Field Capacity (mm) | 200 |
| Infiltration Capacity Coefficient a | 50 |
| Infiltration Capacity Exponent b | 1 |
| Rainfall Threshold (mm) | 1 |
| Soil Capacity (mm) | 400 |
| Initial Storage (%) | 10 |
| Daily Recharge Rate (%) | 25 |
| Daily Drainage Rate (%) | 5 |
| Initial Depth (mm) | 50 |

**Table 21: Music Base and Stormflow Concentration Parameters for Brisbane Catchments**

| Land Use Type | Parameter | TSS ($Log_{10}$mg/L) | | P ($Log_{10}$mg/L) | | N ($Log_{10}$mg/L) | |
|---|---|---|---|---|---|---|---|
| | | Base Flow | Storm Flow | Base Flow | Storm Flow | Base Flow | Storm Flow |
| Urban Residential | Mean | 1.00 | 2.18 | -0.97 | -0.47 | 0.20 | 0.26 |
| | Std Dev | 0.34 | 0.39 | 0.31 | 0.31 | 0.20 | 0.23 |

**Table 22: Water Quality Objectives (from ICC, Ref 2)**

| **Pollutant** | **Reduction in the average annual pollutant load discharging from site** |
|---|---|
| Suspended Solids (TSS) | 80% |
| Total Phosphorus | 60% |
| Total Nitrogen | 45% |
| Gross Pollutants | 90% |

**Table 23: Properties of Rainwater Tanks**

| Properties | Roofwater 1 | Roofwater 2a | Roofwater 2b | Roofwater 2c |
|---|---|---|---|---|
| No. Lots | 106 | 66 | 176 | 78 |
| Low flow by-pass ($m^3/s$) | 0.00 | 0.00 | 0.00 | 0.00 |
| High flow by-pass ($m^3/s$) | 100.00 | 100.00 | 100.00 | 100.00 |
| Volume below overflow pipe (kL) | 318 | 198 | 528 | 234 |
| Depth above overflow (m) | 0.20 | 0.20 | 0.20 | 0.20 |
| Surface area ($m^2$) | 176 | 110 | 293 | 130 |
| Overflow pipe diameter (mm) | 50 | 50 | 50 | 50 |
| Daily demand (kL/day) (10% Av Daily Demand) | 16.615 | 10.345 | 27.588 | 12.226 |

**Table 24: Properties of Bio-Retention Basin**

| Properties | Basin A | Basin B |
|---|---|---|
| Low flow by-pass ($m^3/s$) | 0.00 | 0.00 |
| High flow by-pass ($m^3/s$) | 100.00 | 100.00 |
| Extended detention depth (m) | 0.30 | 0.30 |
| Surface area ($m^2$) | 4245 | 4023 |
| Seepage loss (mm/h) | 0.00 | 0.00 |
| Filter area ($m^2$) | 2091 | 3285 |
| Filter depth (m) | 0.6 | 0.6 |
| Filter median particle diameter (mm) | 0.45 | 0.45 |
| Saturated hydraulic conductivity (mm/h) | 180 | 180 |
| Depth below underdrain pipe (% of filter depth) | 0.00 | 0.00 |
| Overflow weir width (m) | 4.8 | 4.8 |