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CENTRAL HIGHLANDS REGIONAL COUNCIL FLOOD MANAGEMENT REPORT

Section B: Fairbairn Dam – Emerald: Urban Flood Plain Analysis

REPORT PREPARED FOR:
Central Highlands Regional Council

Date:
12 December, 2011

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SUMMARY OF RELEVANT INFORMATION

Project Title	Nogoa Catchment Flood Study Section A: Upper Nogoa – Fairbairn Dam Catchment Assessment and Evaluation
Property Location	Catchments of the Upper Nogoa to Fairbairn Dam, Central Queensland
Project Purpose	Investigate the 2010 Emerald flood event
Applicants Details	Central Highlands Regional Council
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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

1. INTRODUCTION

C&R Consulting were commissioned by the Central Highland Regional Council (CHRC) to undertake a detailed Flood Management Analysis and Evaluation study of the Upper Nogoa River catchment above the Fairbairn Dam and including the Fairbairn Dam Spillway to downstream of the Emerald: Urban Flood Plain. The commission was in response to the re-occurrence of flooding in the Emerald area (i.e. January 2008 and December/June 2010).

The tasks required for the two studies have been developed in response to discussions with the Central Highlands Regional Council and is specifically directed towards the streams and rivers directly impacting on the town of Emerald. The section report, Section B, considers the Nogoa River from the Fairbairn Dam spillway to upstream of the junction with Theresa Creek.

The studies have been undertaken to increase the range of knowledge of the area and ultimately to understand how it functions as a catchment. Once an understanding is gained, management approaches in relation to sufficient notification & evacuation, prevention and reduced damages can be developed accordingly.

1.1 SCOPE OF THE PROJECT

The studies investigate the formation, magnitude, periodicity, and history of events similar to the 2008 and 2010/2011 flood events. This project has utilised a range of different investigative techniques including site assessments, information sessions with local residents and historical groups and a climatological assessment of the region. Other techniques such as analyses of fluvial characteristics of the catchment and the river systems, forensic geomorphic and hydrological analyses and a gaps analysis of current and historical information regarding the Upper Nogoa Catchment were also utilised. Compilation of these soft data has allowed many gaps to be filled and a clearer understanding of the size of the event in all sections of the catchment.

This report is not looking at the Fairbairn Dam itself, and any analysis of this facility and its function or any possible changes to it, is not covered in this report and would require a separate investigation before this can be addressed. This report does however include an analysis of the dam in its current design capacity and the effects that this has had on the flood events described above.

2. BACKGROUND

2.1 CHANGES TO WATERWAYS

Selma Weir was constructed between April 1950 and February1953 and is situated 3.22km upstream of Emerald, and has a storage capacity of 1180 ML. The spillway elevation is EL 170.39 AHD. The crest of the weir is 7.32 m above foundations and has total length including cut-off walls of 192 m.

Fairbairn Dam also known as Lake Maraboon was constructed between 1968 and 1972. The structure was not designed specifically for flood mitigation purposes but as a water conservation effort for urban and irrigation supply with a capacity of 1.3 million ML, it does however have mitigation qualities. The spillway is situated 19km upstream of Emerald with water from the spillway taking 4-5hrs for the peak flow to reach the Vince Lester Bridge. The spillway was designed for a flood flow of 15600 m^3/s, which is more than triple the flow of the 2010/11 event at 4330 m^3/s and 5 times the 2008 event (2952 m^3/s).

Two constructed open-earthed channels, Selma and Weemah Channels act as delivery systems from the Fairbairn dam for the surrounding Emerald Irrigation Area (EIA Selma has a maximum release of 770ML/day, Weemah with a maximum release of 330ML/day They were built in conjunction with the dam (which has a maximum river release of 1500ML/day). The Selma Channel runs for approximately 47 km on the western side of the Nogoa River and services farms to the north and west. The channel is comprised of one main channel and a number of subsidiary channels, which are fed by gravity providing the dam remains above 68% capacity. The Weemah Channel services the farms to the east with a channel length of approximately 53km. Water is fed in to this channel via a 6m diameter tunnel with an intake tower on the upstream side of the dam wall.

The EIA drains an extensive surface runoff system of approximately 207 km in total length. This system includes two larger channels designed, for a 1:10 event, that affect flood water in and around the township of Emerald; LN1 and LN3. The LN1 channel drains 2330 ha of irrigated cropland before entering Emerald where it acts as an open storm water drain before entering an anabranch of the Nogoa River to the north of the town. The LN3 channel enters the Nogoa River just above the town weir and has a total catchment of 4280 ha. These two drainage channels act as major flow paths in the case of flooding due to their drainage of flow across the flood plain. The EIA also includes other overland flow dams and diversion streams that have been constructed over time however these have less effect on the flow in and around the town.

2.2 CLIMATE

The climate of the Central Highlands is typical of the Seasonally Arid Tropics, where rainfall is strongly seasonal, highly erratic both in duration, intensity, and periodicity, and where the majority of the summer rain falls within a very short period (occasionally within a few days) followed by extended periods of relatively little rainfall. Storm rains at the beginning of the summer wet season are sudden and intense and often restricted to isolated falls within individual catchments. Sunlight hours are high with sunny days usually in excess of 300 days per year. Evaporation is high, exceeding precipitation for approximately 80% of most years.

The combination of strongly seasonal rainfall, high sunlight days, and extreme evaporation creates an environment where the soils dry out rapidly. Under these conditions the landscape is dry and parched by the beginning of the first storm rains of the season.

Runoff is rapid across the unprotected landscape. Dependent on where the rains fall, flow through the river channels is equally rapid.

Summer rainfall in the Central Highlands Region is driven by the location of the Inter Tropical Front, the North East Australian Monsoon trough and/or cyclonic activity. The pathway of the rainfall from development to the Central Highlands is dependent on the source of the event. In general, while the rainfall pathway is relatively well defined, the location of the rainfall within the band of activity is volatile, falling as isolated cells that may locate over one area for days.

The area is a zone of regular interactions between the seasonal summer rainfall associated with the Inter Tropical Front, the North East Australian Monsoon, and seasonal cyclonic activity, with the zone which receives appreciable winter rainfall associated with the tracks of low pressure systems sweeping across from the west and south west of Australia. Typically the balance between summer and winter rainfall is approximately 70:30. Intense rainfall events frequently occur when the warm tropical air intersects the cold frontal area coming up from the south. Thus, intense rainfall events are more likely to occur early in the wet season (December / January, e.g. 2008 and 2010), or late in the wet season (e.g. April 1990).

The topography of the Central Highlands region is such that it tends to guide weather events from both north and south so that the interaction points between warm air from the north and cold air from the south fall within the area. Two such areas that both feed tributaries of the Nogoa River, and thus are relevant to the Emerald district, have been identified as “hot spots” for intense activity in the Central Highlands Region.

Therefore, understanding the climate and the movements of the rain events is critical to develop probability of river flows and the adjoining flows which make for the large events if/when synchronous. If the rainfall was more regular (e.g. temperate climates where the rain falls fairly evenly throughout the year) the magnitude of the flooding event would be considerably less. This, in part, is due to the nature of the soils whereby under arid conditions impermeable soil crusts may develop which, in extreme rainfall events, lead to a runoff coefficient close to one (1).

In the Central Highlands region, rainfall and river discharge data are sparse and of short duration (often less than 50 years). Certainties in modelling data are weakened by the lack of data and modellers have resorted to the inclusion of data from overseas to extend the periodicity of the records. In temperate climates this fusion of data has a reasonable chance of prediction with a reasonable degree of certainty. In the seasonally arid tropics, the climatic conditions bear no similarity to temperate climates. Rainfall intensity, duration, and periodicity are erratic, often restricted to a few short months of the year, with rain falling as a series of intermingling cells across, or within, a catchment. Evaporation exceeds precipitation for approximately 90% of the year, often including days recording periods of heavy rainfall. Sunny days usually average a minimum of 300 days per year. Summer rains begin suddenly and runoff over the parched earth can be as high as 100% in the initial stages. Heaviest rains are associated with cyclonic and/or monsoonal activity and each event may last for weeks, with the intensity and duration of the rainfall varying continuously. Flood Prediction Models derived from temperate climatic data cannot adequately resolve the disparities in the tropical climatic data.

Thus the impacts of flooding are not only socially disruptive, but have a huge cost on the overall economy. It is therefore essential that infrastructure associated with communities and private enterprise is realistically aligned to the levels of risk associated with these events.

3. METHODOLOGY

3.1 HISTORICAL DATA

The premise of this study is the analysis of the Emerald Urban Floodplain in the wake of flooding events throughout the township and surrounding districts in both 2008 and 20010/11. It is important to identify within this scope the event that is the basis of the analysis. Comparison of the hydrographs of the 2008 and 2010/11 events (Figure 1) shows that the 2010/11 flood height was approximately 0.69 m higher at the peak of the flood through the Vince Lister Bridge than the 2008 event. As the 2010/11 event has the higher record of the two events then this is the event that the modelling is based upon.

During the 2010/11 event the Fairbairn Dam acted as a buffer and reduced the discharge by 26%. In 2008 it reduced the discharge by 30%.

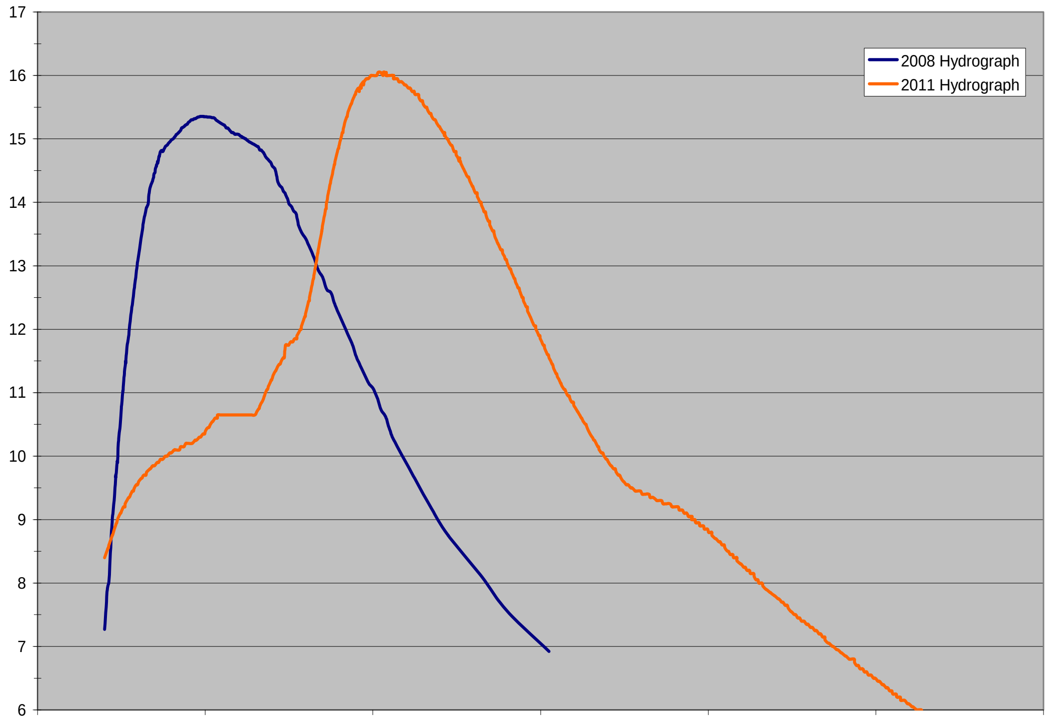

Figure 1: 2008 vs 2010/11 Hydrograph

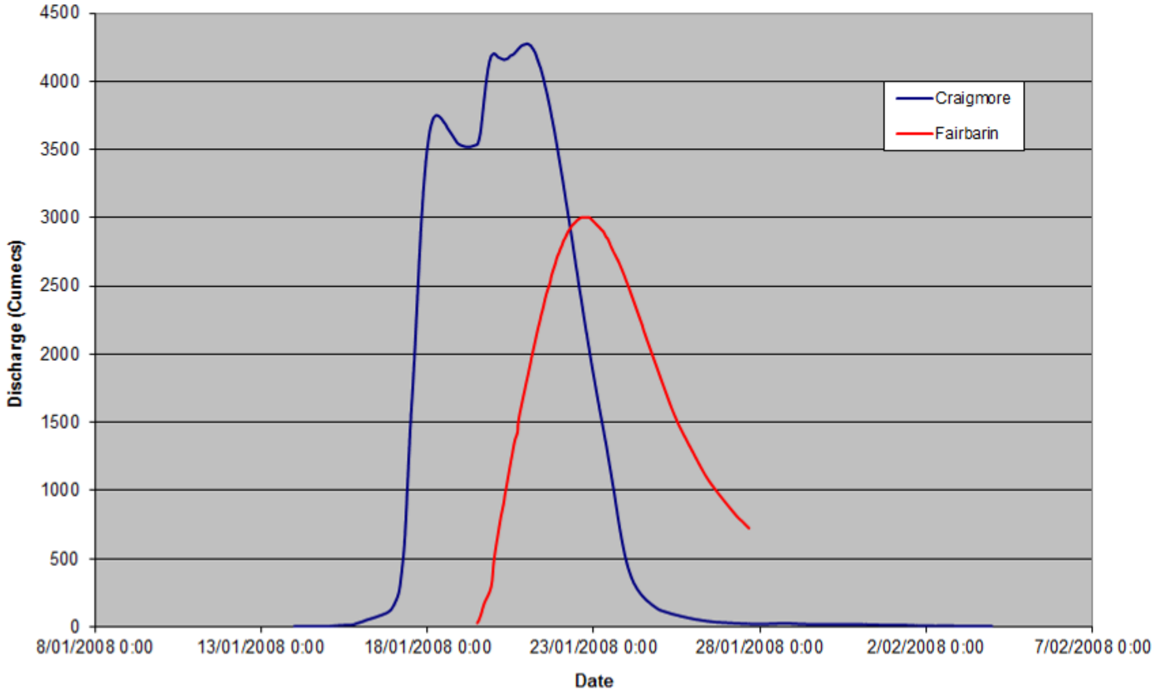

Figure 2: 2008 Event Hydrograph

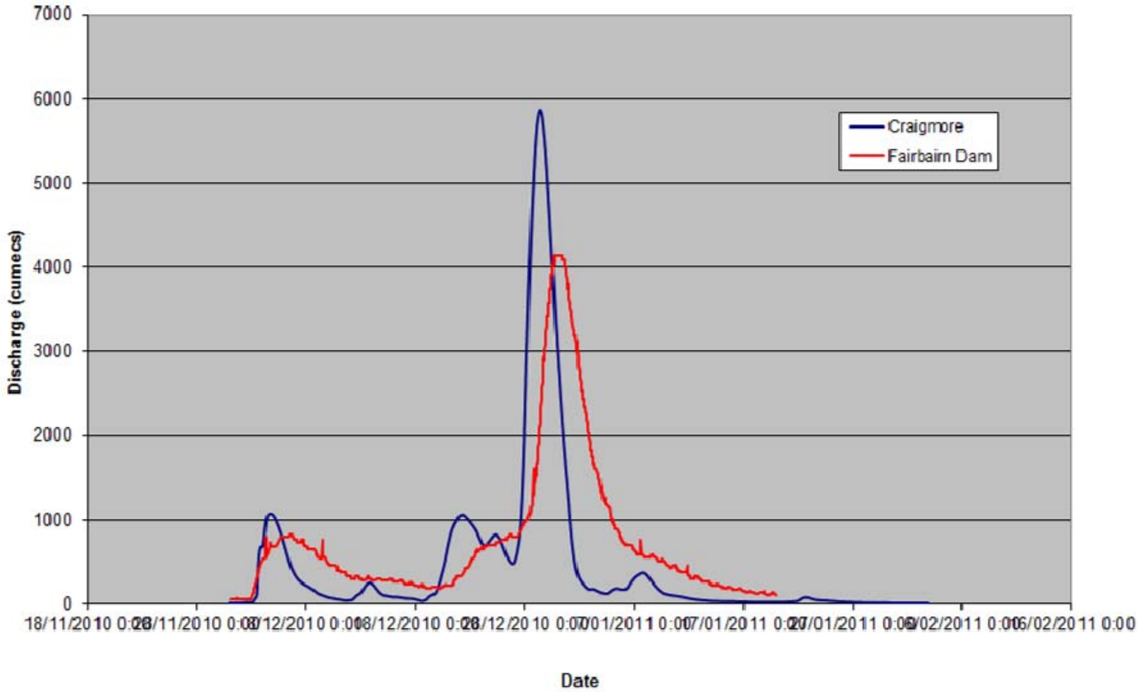

Figure 3: 2010/11 Event Hydrograph

3.2 TOPOGRAPHY

A three-dimensional LIDAR dataset of the region surrounding Emerald was provided to C&R Consulting for the purposes of this assessment. The dataset consisted of a series of contours as well as a series of 1 km x 1 km ESRI Digital Elevation Models (DEMs) at a resolution of 1 m x 1 m. This data was used to generate topography for the modelling. The methodology employed is outlined below:

- The data was imported into ArcGIS 9.3.1 as the series of ESRI DEMs and was merged into one continuous DEM from the Fairbairn Dam down through the township to the junction with Theresa Creek using 3D Analyst.
- This 1 m x 1 m DEM was then re-sampled at coarser resolutions to aid in calculation times for the model. The coarser resolutions of the DEMs include 2 m x 2 m, 5 m x 5 m, 10 m x 10 m, 20 m x 20 m, and 50 m x 50 m.
- The various resolution DEMs were exported as ASCII's text files for importation into the modelling software.

3.3 SURFACE WATER FLOODING

3.3.1 MODELLING

The modelling engine TUFLOW was used to mimic the 2010/11 event in order to make changes to the infrastructure within the model and assess the potential effect these changes have on the water heights and velocities throughout the modelled flow. TUFLOW is a modelling package developed by BMT WBM and the University of Queensland specifically for Australian conditions. The model does not include a Graphical User Interface, however one has been developed by Aquaveo called the Surface Modelling System (SMS) which allows GIS data and CAD topography to be used to construct and analyse the models.

TUFLOW supports the creation of one-dimensional models[1], two dimensional models[2] and integrated one and two dimensional models. A two dimensional model was chosen to represent conditions on-site as this more reflects reality and for ease of use. Where blockages or obstructions occur in the waterway (such as culverts) these were represented as a one dimensional model within the two-dimensional model's extents and conditions.

3.3.2 MODEL PARAMETERS

3.3.2.1 *TOPOGRAPHY*

The greater Emerald region, from the dam down through the town, was represented by a 2500 m^2 grid (50 m x 50 m grid cells) where each cell represented a height in metres AHD. Around the township of Emerald a finer resolution grid was required to successfully model water flow interaction with the built environment and this area was represented by a 100 m^2 grid (10 m x 10 m grid cells). Finally, to address restrictions in flood level water flow around the bridges across the Nogoa River, a higher resolution 25 m^2 grid (5 m x 5 m grid cells) was used to represent this region. All grid values were generated from topographic datasets as outlined in Section 3.1.

[1] One dimensional models use a series of cross-sections to determine water level height and velocity. They are well suited for applications where flows occur in the channel but are not suited for overland flooding as water can not 'move' laterally in the models.
[2] Two dimensional models use a topographic dataset (usually generated by LIDAR with survey information added to the dataset) to model water 'movements' across a land surface.

3.3.2.2 *STRUCTURES*

Culverts in a TUFLOW model are represented as one-dimensional models nested within the two-dimensional model extent. Flows were transported from the two dimensional model into the one dimensional model (in the vicinity of the culvert), routed through the structure and flows were then re-distributed into the two-dimensional model. Culverts were only used within the recommendations for mitigation section of the modelling analysis for the section to the west of the Vince Lester Bridge.

Bridges are represented in a TUFLOW model by flow restrictions that mimic the disruptions to water flow caused by the pylons and crest of a bridge. Flow restrictions were used within this Emerald model for the bridges at the John Gay Bridge, the Central Railway Bridge and both the Old and New Highway Bridges. The details of the Railway Bridge were obtained from construction designs delivered by the CHRC, whilst the heights and makeup of the other bridges were estimated by using nearmap.com aerial imagery, photographs and the LIDAR topography supplied by the CHRC.

Other drainage infrastructure such as Selma Weir, the SunWater Drainage system, and CHRC drainage structures were not included in the model as these were picked up in the Department of Environment and Resource Management (DERM) LIDAR topography provided by CHRC.

3.3.2.3 *UPSTREAM BOUNDARY CONDITIONS*

The upstream boundary conditions for the model were set as an input hydrograph (discharge vs time) just below the Fairbairn Dam. After some experimentation with the dynamics of the TUFLOW model, it was decided not to use the Fairbairn Dam hydrograph as the upstream boundary conditions as the rapid fall of the dam slipway made the model unstable. Thus the flow rates of the Emerald Alert hydrograph from the 2010/11 flood event were used.

3.3.2.4 *STARTING CONDITION*

Flood modelling requires that the worst-case scenario is considered in order to provide adequate mitigation measures against potential flooding. Using this assumption the river system was assumed to be full at the beginning of the model in order to model the event within a full system. To achieve this, 1500 cubic m/sec were allowed to flow from the upstream boundary of the model domain through the system before the 2010/11 event hydrograph was run within the model. The event was then run over a total of 80 hours

3.3.2.5 *DOWNSTREAM BOUNDARY CONDITION*

The downstream boundary condition was set as a computed ratings curve (HQ curve) which is used to calculate the water depth at any cell based on topography and an anticipated water surface gradient. An assumed water level gradient is required to be input into the model so that it can calculate the expected height and velocity of flow across the boundary condition based on the volume of water reaching it. A gradient of 0.00041m/m was used. This value was the average slope of the channel six transects taken in the area of the downstream boundary condition.

3.3.2.6 *ROUGHNESS VALUES*

TUFLOW uses Manning's Roughness values to calculate the resistance exerted on the flow from vegetation, stream sinuosity and resistance from structures. TUFLOW can use depth-based Manning's N values in an attempt to more closely represent reality. This process attempts to mimic the reducing resistance to flow in deeper flow paths. Manning's N values can be specified for two depths, and linear interpolation is used to determine the Manning's N values between these depths.

Land uses were digitised in ArcView 9.3.1 based on the aerial photograph provided in NearMap (http://www.nearmap.com). These land uses were then used to map Manning's N coefficients, which are outlined below in **Error! Reference source not found.**.

Land Use	Manning's N		Photo Pattern
	1m Depth	3m Depth	
Bare Ground[3]	0.03	0.009	
Buildings	20	n/a	

[3] This land use is the default setting for areas without a prescribed land use

	Manning's N		
Croplands	0.05	0.0175	
River	0.001	0.0005	
Shrubbery	0.09	0.0225	

	Manning's N		
Water Storage	0.001	0.0005	

Table 1: Manning's N values for various land uses across the modelled catchment

3.4 MODEL SENSITIVITY ANALYSIS

Once the model was initially set-up and all flood mitigation scenarios were identified, a number of parameters were varied in the model and each scenario re-run. This was performed to both maximise the accuracy of the model outputs to the mapped extent of the 2010/11 flood event and to evaluate the effect of changing model parameters on these results.

The parameters which were adjusted include:

- Downstream Boundary Conditions. The slope of the water surface leaving the model was adjusted between 0.0001 (the lowest gradient found in the floodplain) and 0.001 to evaluate these effects on the model.
- Manning's N Values. Manning's N values were input into the model as a series of roughness vs. depth curves. The values were increased by approximately 20% and the model base-scenario (without mitigation measures) was re-run to determine the effects of variations to Manning's N values to water levels and velocities.

The results of the sensitivity analysis revealed limitations in the model including the necessity of including the pedestrian underpass under the railway between Sullivan St and the Capricorn Highway opposite the Car Spa. Culverts and underpasses were not included in the overall model due to model stability constraints, although many of these features were modelled independently of the dam down model.

4. RESULTS

4.1 UPPER CATCHMENTS RUNOFF INTO FAIRBAIRN DAM

Runoff amounts were calculated for the Zones 1 to 4 using a range of empirical relationships for soil types, slopes, pre-existing conditions, rainfall and rainfall intensity. These empirical relationships have been derived over the last several years from numerous studies in the Central Queensland region.

A 72hr event of 5mm/hr was modelled, this rate being the C&R estimate of the occurrence of a real event probability 0.01 in any one year. This contrasts with the BOM estimate for a similar probability of 4.22mm/hr over 72hrs. This difference arises from the use of a Power Frequency Distribution rather than the Log Pearson Distribution used by the BOM. From investigations throughout Tropical Queensland, the general relationship between the BOM calculated event values using Log Pearson Distributions and values calculated from the superior fitting Power Frequency Distribution is a multiplier of approximately 1.3. For this investigation, the value of 5mm/hr for 72hrs, although only x1.2 the BOM value, was chosen because of its numerical simplicity. It is considered that if the BOM values are used directly, then there is a certainty of 50% to 55% that the event modelled will be below the flow level calculated.

Three scenarios were modelled:

1. The Event was over the whole area for the entire duration.

2. The Event moved sequentially from Zone 4 to Zone 3 to Zone 2 to Zone 1 at a speed synchronous with the times of concentration for the individual catchments.

3. The Event moved from being stationary over Zones 4 to 2 and then moving to Zone 1 at a rate synchronous with the occurrence of a maximum flow over the Fairbairn Dam spillway.

Three estimates of runoff were made:

1. At a certainty of 68.2%, that the 0.01 probability event modelled would be below this level.

2. At a certainty of 95.4%, that the 0.01 probability event modelled would be below this level.

3. At a certainty of 99.6%, that the 0.01 probability event modelled would be below this level.

It should be noted that these are the conventional one, two and three sigma values that strictly relate only to a standard normal distribution. Since the underlying distributions for large rainfall events are significantly right hand skewed, these certainty levels are themselves somewhat uncertain. The value of 68.2%, lying within the central portion of the distribution is likely to be a reasonable estimate. With the assumption of a Power Distribution Frequency curve, the 95.4% and the 99.6% values are likely to be slightly overestimated and thus are on the conservative side. This conservatism is considered preferable to the underestimation of these values that may occur if a Log Pearson type of distribution is used. Values for these scenarios are tabulated in Table 2.

Scenario		Flow Estimate for 68.2% certainty (m^3/sec)	Flow Estimate for 95.4% certainty (m^3/sec)	Flow Estimate for 99.6% certainty (m^3/sec)
Zone 4	Individually	5389	10,015	13019
Zone 3	Individually	5600	10,570	13741
Zone 2	Individually	4375	8260	10739
Zone 1	Individually	2336	4419	5745
Stationary over Zones 1-4 for 72hrs. Total flow at Emerald Downstream Point.		9591	18091	23519
Stationary over Zones 2-4 for 72hrs. Total flow at entry to Fairbairn Dam.		9538	17991	23389
Stationary over Zones 2-4 for 72hrs, then moves synchronously with flow to Zone 1. Total flow at Emerald Downstream Point.		11875	22395	29114
Stationary over Zones 1-4 for 72hrs, then moves synchronous with flow to Zone 1 at same rate as flow maximum. Total area ratio method. Total flow at Emerald Downstream Point.		18011	33960	44148
Event moves synchronously across Zones 4 to 1 at same rate as flow maximum. Individual area method. Total flow at Emerald Downstream Point.		17702	33376	43389

Table 2: Estimation of Flows in Emerald Area Assuming 5mm/hr, 72hr Event

4.2 FLOWPATHS

The modelling of the 2010/11 flood event that moved through Emerald highlighted a series of flow pathways that caused the flooding through the township. Many of these flow pathways are known to local stakeholders and were identified during the flood event in a series of photographs taken leading up to the peak, during the peak of the floods and after the peak however the numerical outputs of the modelling of the flood event outline these outputs definitively.

There are two classifications within the flow paths identified by the modelling, using depth and velocity of the flow; the main flow paths and the secondary flow regions. Areas within the model that have no current development and a flow above 1 m/sec are considered to be a main flow path, with regions that show less than 1 m/sec flow and a water depth of less than 0.5 m are considered secondary flow regions.

The main flow paths through town, identified within the peak flow of the modelled flood at a velocity of over 1 m/sec, are show in

with the blue shading.

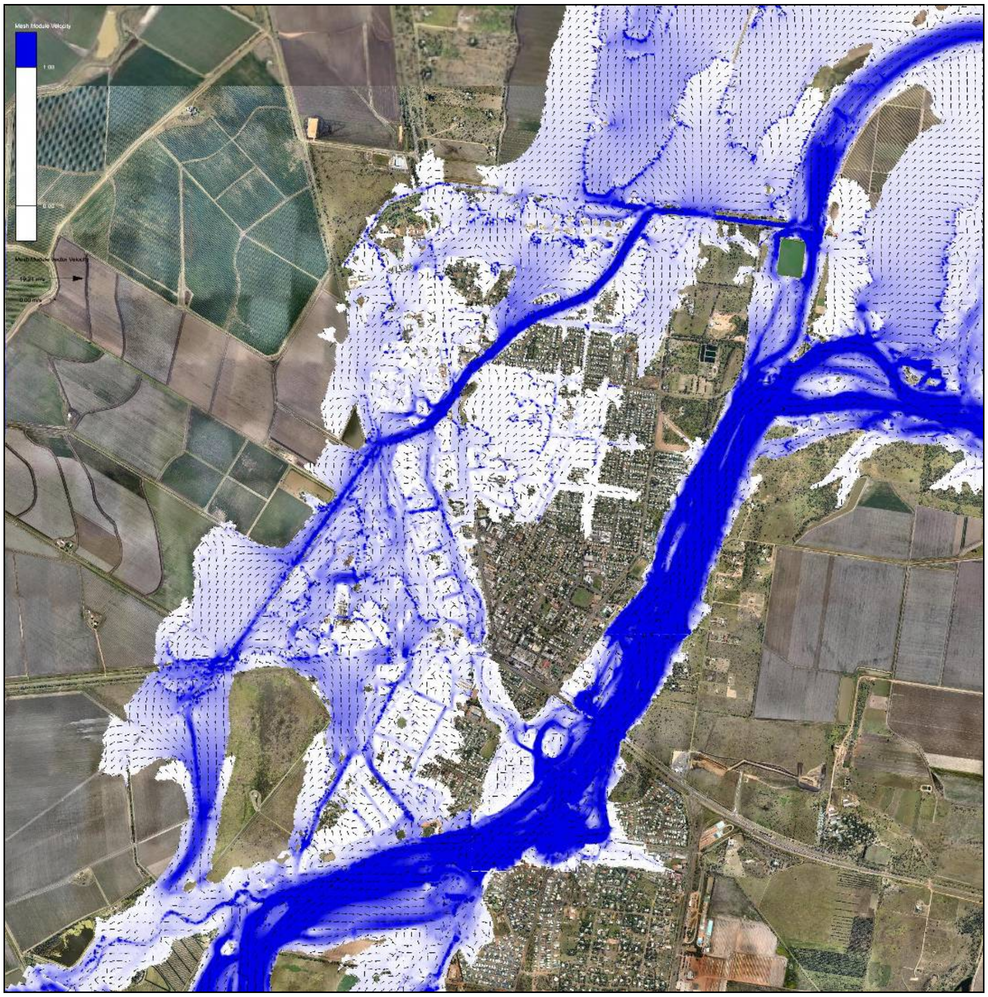

Figure 4: Main flow paths identified within model

The recommendations in this report are designed to enhance the current flow paths to reduce pooling and afflux but at the same time C&R are recommending the protection of the flow paths that are displayed in figure 2. The dark blue areas are to have total protection moving forward and the lighter blue areas are to be treated as secondary flow path areas that could be developed as long as 50% of the area is left as natural surface with no impediment to flow.

5. DISCUSSION

To effectively utilise the results of this study, a better understanding of Annual Exceedance Probability (AEP) and Average Recurrence Interval (ARI) for this area is required to limit any misgivings the terms may have.

5.1 ARI / AEP

5.1.1 Annual Exceedance Probability (AEP)

Annual Exceedance Probability (AEP) is the probability of rainfall events of particular magnitude occurring within a given period. As such, low probability events are essentially derived from right hand skewed frequency distributions. The matching of particular rainfall events with a temporal probability relies, almost totally, on the right hand tail of the distribution. Some distributions will underestimate the rainfall associated with, for example, an AEP of 0.005 (i.e. a 1:200 year ARI) whereas others will overestimate the rainfall associated with a similar AEP. From the short rainfall records available throughout the area, accuracy in AEP estimations is very difficult. For example, approximately a 300 year record is required to give a 5% uncertainty in an AEP of 0.01 (i.e. a 1:100 ARI).

Records of this length are not available in Australia. Consequently, accuracy in probability estimation is difficult to achieve due to both the length of record and the true underlying frequency distribution pertinent to the area under investigation. For International comparative purposes, a Log Pearson III distribution is used in Australia and some other countries. The Log Pearson III distribution is known to underestimate rainfall magnitudes of low probability events. Underestimation of rainfall associated with low probability events may be interpreted as a greater frequency of actual occurrence.

For the Central Highlands, even with the short duration of records available, it is likely that a Power Frequency Distribution will more accurately reflect the nexus between rainfall magnitudes and probabilities than the Log Pearson III distribution. Use of a Power Frequency Distribution will provide a more conservative answer with respect to AEPs and thus give a greater degree of protection for any given AEP.

5.1.2 Average Recurrence Interval (ARI)

Average Recurrence Interval infers that an event of a designated magnitude will occur within a given period (e.g. a 1:100 event implies that it will occur once every one hundred years). An AEP of 0.01 implies that in any given year there is a 1% chance of its occurrence. Average Recurrence Interval is a misleading term and should be dropped in favour of AEPs in spite of the attendant difficulties associated with AEPs.

The shortness of data records place similar constraints on ARIs to those experienced with AEPs and this influences public perceptions with respect to the precision, accuracy, and uncertainties associated with both measurements.

It is recommended that CHRC stipulates limitations and ruling that protect all stream flow paths including out of bank flow paths that are left throughout the flood plain within the town area. These flow paths must be clearly identified, mapped, maintained and protected. To achieve the best possible result; other Government organisations including Queensland Rail, Main Roads and SunWater must work with Council to develop, protect and maintain these flow corridors.

The move throughout the state at present is for the building heights for towns to be set above the highest recorded flood, which in Emerald is the 2010/11 flood event. The 2008 flood event qualifies as the 1:100 event for Emerald, using the Bureau of Meteorology (BOM) designation. Due to the uncertainties outlined above, there is currently a review being undertaken on using ARIs as official designations. By using statistical methodologies devised by C&R Consulting (C&R Consulting 2010, Appendix 1), the 1:100 ARI event would be equivalent to around 30% above the BOM 1:100 ARI. This is very close to the level of the 2010/11 event which is an additional reason that the 2010/11 event was the event selected to be modelled. It is interesting to note that there could be a policy for building at the highest recorded flood level, but then have critical infrastructure like roads and bridges using the 1:100 height that has the ability to impact the buildings that are built at the highest flood height.

5.1.3 EMERALD HYDROGRAPH

There exists an insufficient amount of data to establish a ratings curve for the Emerald gauging station, as volumes are currently unable to be measured onsite. Full ratings curves for two stations upstream (Fairbairn Dam and Craigmore) are available. These can be used to calibrate the river height data for Emerald to estimate a flow volume.

It has been observed that the length of time it takes for water to travel between the Fairbairn Dam and Emerald is approximately five hours. To calibrate the Emerald station, it was assumed that the volume of water flowing at the Emerald station was exactly the same as the amount flowing at the Fairbairn Dam five hours ago. Using a years worth of data, a number of points were collected. In order to prevent an unfair weighting of points in one particular area, the sampling of the dataset was conducted at random, ensuring that the highest and lowest values were included in the data.

These data points are shown in Figure 5, with a Power Curve Regression performed to produce the curve.

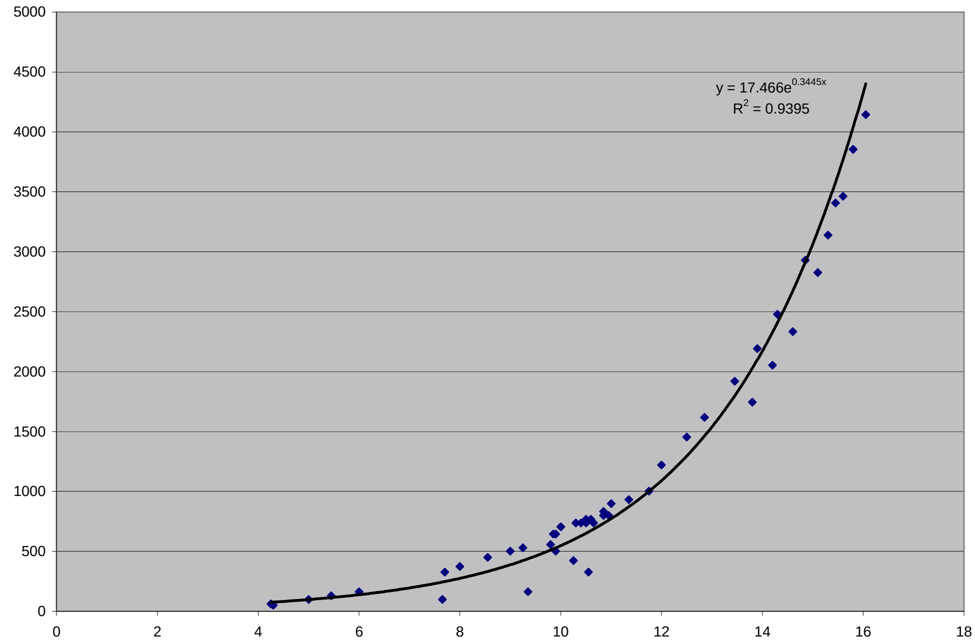

Figure 5: Predicted Ratings Curve for Emerald

This method is likely to be subject to a fair amount of uncertainty, due to the potential for water to be added or removed between the Fairbairn Dam and Emerald. Regardless, this distance is not especially large to warrant a massive deviation from the prediction. This is shown in the graph above, where most of the data points were approximately on the curve. The regression coefficient r^2 for this dataset was 0.9395, which means that this curve approximates the data very well.

From this approximation a ratings table can be obtained, as shown in Table 3.

Height	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
3	-	-	-	-	0	2.3	8.8	18.6	31.9	48.5
4	69.3	75.3	77.8	80.5	83.2	86.0	89.0	92.0	95.1	98.4
5	101.7	105.2	108.8	112.5	116.3	120.3	124.4	128.6	133.0	137.6
6	142.2	147.1	152.1	157.3	162.7	168.2	173.9	179.9	186.0	192.3
7	198.9	205.6	212.7	219.9	227.4	235.1	243.2	251.4	260.0	268.9
8	278.0	287.5	297.3	307.4	317.9	328.8	340.0	351.5	363.5	375.9
9	388.7	402.0	415.7	429.8	444.5	459.6	475.3	491.5	508.2	525.6
10	543.5	562.0	581.1	600.9	621.4	642.6	664.5	687.1	710.6	734.8
11	759.8	785.7	812.5	840.2	868.8	898.4	929.0	960.7	993	1027
12	1062	1098	1136	1175	1215	1256	1299	1343	1389	1436
13	1485	1536	1588	1642	1698	1756	1816	1878	1942	2008
14	2076	2147	2220	2296	2374	2455	2539	2625	2715	2807
15	2903	3002	3104	3210	3319	3432	3549	3670	3795	3925
16	4058	4197	4340	4488	4641	4799	4962	5131	5306	5487
17	5674	5867	6067	6274	6488	6709	6938	7174	7419	7671
18	7933	8203	8483	8772	9071	9380	9699	10030	10372	10725
19	11091									

Table 3: Predicted Ratings Table for Emerald

6. RECOMMENDATIONS

6.1 PREPAREDNESS

6.1.1 RIVER GAUGE RATINGS CURVES

The development of a policy for reporting and recording gauging stations where there is some concern over their accuracy is urgently required. This is highlighted by the river gauge at Retreat Theresa Creek at Gregory HWY, this gauge was reviewed as unreliable in high flows in 1995/6 by Natural Resources but still continued to be used by Government agencies up until 2008. This river gauge was underestimating flows by over 100%. During the last two floods a large percentage of river heights exceeded the ratings curves for the Gauging stations making it very hard to establish flow volumes. A lot of the river gauges are not accessible during high flows for boat crossings to correct ratings tables but detailed surveys could be established to extend these ratings curves beyond the heights recorded in 2010/11. LIDAR is used by the mining industry throughout this area and would give a very good survey through these river gauges.

The development in and around existing river gauges has to be assessed for the perennial impact on the accuracy of the gauge. If the development is shown to have an impact and cannot be shifted the developer must pay for the relocation of the gauging station and the recalibration of the data required because of the shift.

6.1.2 LANDHOLDER GAUGE BOARDS

At present there are gaps in data collection and monitoring leading up to and during events in the region. By filling these gaps through more extensive monitoring of streams during events will aid in the timeliness of preparation. It will also allow better determination of the sizes of the event.

To do this, installation of manual gauge boards within the upper and sub catchments of the Nogoa is recommended. The identification of the position of these sites is based on the proximity of homesteads to the streams to enable safe reading and reporting during events. It is recommended that the gauges on the gauge boards be marked with the 2010/11 flood height as 0 with + and - marking from there with the exception of Mowbray homestead for the Medway creek. The 0 mark for this gauge should start at the 2008 flood height as this is the highest level recorded. Location of these gauge boards should include;

- Nogoa River at the bridge crossing to Telemon homestead.
- Wharton Creek at the new Wharton Creek homestead. Claude River at Mantuan Downs homestead.
- The Medway at Rutland homestead.
- The Medway at Mowbray homestead.
- Cona creek at Cona Creek homestead.
- Vandyke Creek at Euneeke homestead?
- Nogoa River and Vandyke Creek junction at the Eumara homestead.
- Nogoa River at Raymond homestead.
- Nogoa River at Nandowrie homestead.

These gauge boards will not only assist the town of Emerald during a flooding event but the land holders and new residence or owners in the area to understand the risks.

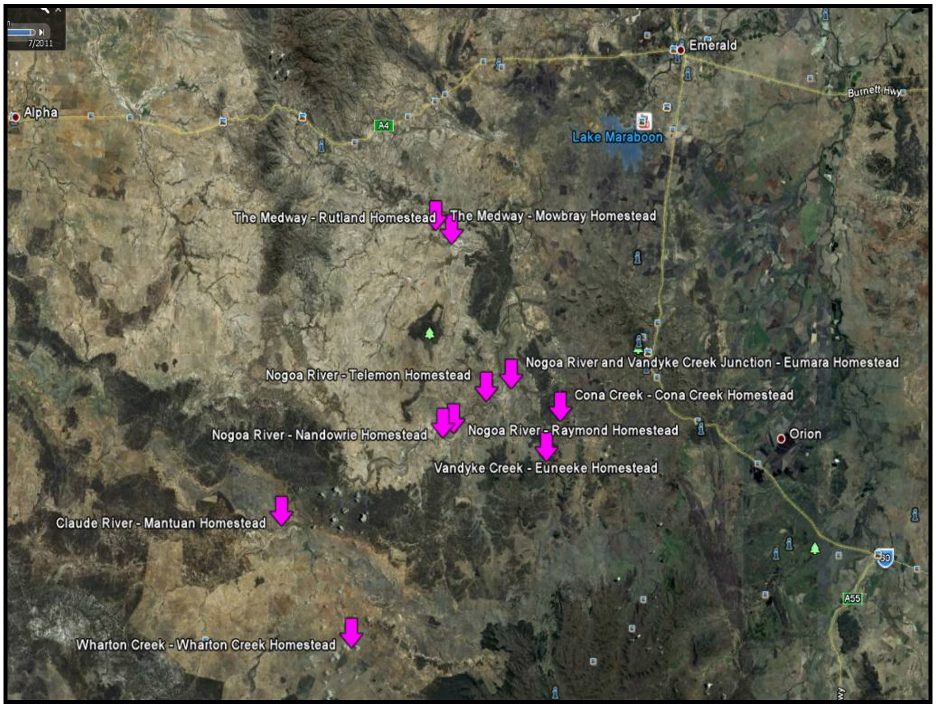

Figure 6: Recommended landholder manual gauge boards

6.2 TOWN PLANNING PROTECTION FOR OUT OF RIVER BANK FLOW CORRIDORS

6.2.1 EARLY WARNING RAINFALL RUNOFF MODEL

The development of a Rainfall runoff model for the Catchment above Emerald for medium to large rain events enables a prediction of stream flows, heights and the timing of those flows at points throughout the catchment. This model could be produced as a simple spread sheet form to be run during or at the end of a rain event to give the earliest possible prediction to the community downstream, including the township of Emerald. Used in parallel with the gauge board readings (at the top of the catchment as points of truth), this model will lead to a very accurate prediction of event size. The model uses rainfall runoff coefficients in medium and large events as well as the regulating ability of the streams at different flows to determine the volumes, heights and timings throughout the catchment.

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To do this accurately, a better understanding of Annual Exceedance Probability (AEP) and Average Recurrence Interval (ARI) for this area is required to limit any misgivings the terms may have.

6.2.2 THE ALERT NOTICES BY SUNWATER TO BE PREDICTED SPILLWAY HEIGHTS NOT JUST RECORDED

Currently SunWater sends out notices of dam spills and heights of spill to all land holders immediately downstream of the dam after it has occurred. These notifications must include predictions of maximum height of the event for the notifications to be of any use to the landholders and residents downstream.

6.2.3 THE PLACEMENT OF A GAUGING STATION ON VANDYKE CREEK

The placement of this gauging station will record and monitor stream flows from a large part of the sub- catchment currently not monitored. This will mean another 12% of the catchment will have an actual flow volume recording 3 days out from reaching Emerald.

6.3 STRUCTURAL MITIGATION

The structural recommendations have been based on modelling results, on ground data collected during the last two events, historical data, residence information and a large series of photos leading up to, during and after the events. All relevant flow structures around town were located, evaluated and measured in comparison with flow volumes in each area. Appendix 2 reviews images of each flow structure, their sizes and locations. This will be relevant for the structural mitigation recommendations.

6.3.1 INCREASED FLOW PATH UNDER NOGOA RIVER RAIL BRIDGE EMERALD

Increasing the flow path by placing large box culverts or small bridges on the western side of the main bridge up to Opal Street will add as much as 20% to the capacity of the main channel of the river in this area. This will allow the flow to move through the town much faster and reduce pooling.

C&R Consulting have modelled the effects of an increased flow path under the Rail Bridge by inserting either a small bridge or 8 culverts under the Vince Lester Bridge, increasing the capacity by an additional 20% to the main channel. Each culvert within the model represents five 1.5 m x 1 m rectangular box culverts, located in the positions indicated in Figure 7. The effect that this has on the flow can be seen in Figure 8 and Figure 9. Figure 8 shows the current situation where the ballast of the railway bridge acts as a levee bank holding back the flow, with pooling increasing the depth of the flow (darker blue) until it spills over the bridge. This also has the effect of not allowing the water to clear downstream of the bridge as quickly and creates flow towards the river along the ballast towards the river on the upstream side.

Figure 7: Culvert locations under western side of the railway bridge

Figure 8: Modelled 500 m3/sec test flow with no culverts

Figure 9 shows exactly the same modelled flow with the culverts included, and shows a shallower depth on the upstream side of the railway bridge as the flow goes through the culverts and clears the downstream area along the golf course. The flow through these culverts also prevents floodwater from going over the highest section of the bridge in this model which has a much higher flow rate than was experienced in the 2010/11 event.

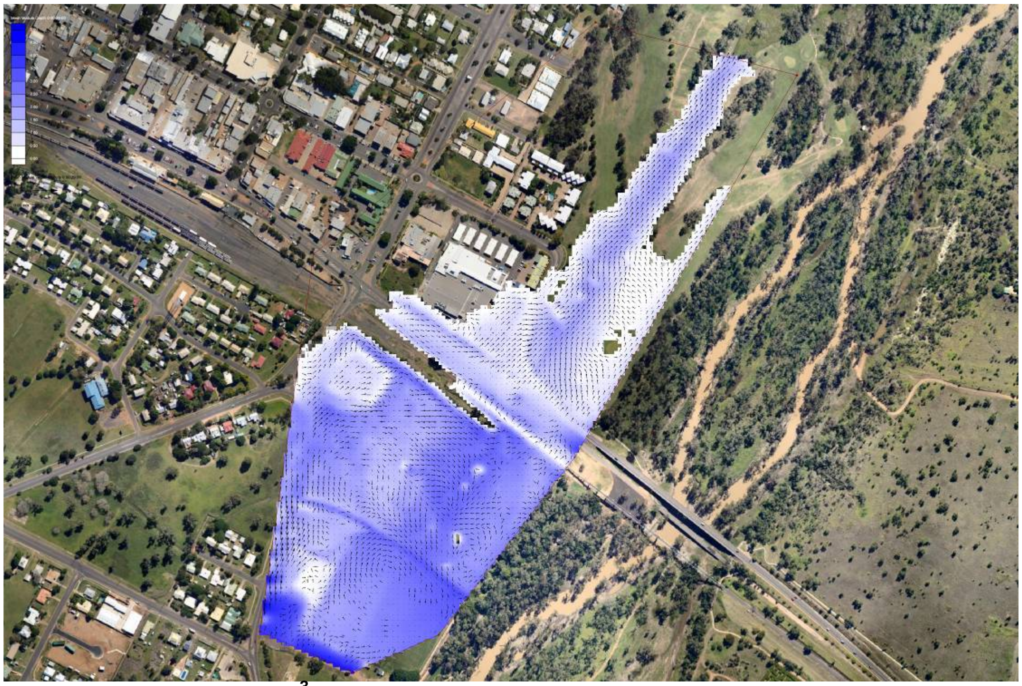

Figure 9: Modelled 500 m^3/sec test flow including culverts under the levee

6.3.2 ELEVATION OF NEW STREET

The old flow path that comes out of the river across New Street, the QRI sports ground then through the railway pedestrian under pass can be controlled by elevating New Street. This flow path is restricted by the pedestrian walk way at present as well as the sports grounds (previously swamp land) downstream. It is C&R's recommendation that New Street be built up to 178.39m AHD which is 1.47m above current height with two 750mm pipes at ground level; reducing impacts of access across town and the possibility of the rail line over topping through the centre of town.

The elevation of 178.39m is important as it allows flows from the west and south of the rail line to make its way back into the river over New Street. This would have seen the road inundated in 2010/11 but not in 2008. This project cannot be done without increasing the capacity of flow under the rail bridge.

Dimensions relating to New Street and flood heights	
Current New Street	AHD 176.92m
2008 flood height at New Street	AHD 178.04
2010/11 flood height at New Street	AHD 178.69

Table 4: Dimensions Relating to New Street and Flood Heights

6.3.3 INCREASED FLOW IN LN1 UNDER GREGORY HIGHWAY

The current Gregory Highway crossing over LN1 consists of 5 x 2.1 x 2.1 RCBC's which have 33% less flow capacity than the crossing up stream on Hogans Road. The recommendation is to double the capacity under the Gregory Highway by adding 3 new 2.1 x 2.1m RCBC's as well as 8 new 1.2 x 1m RCBC's or their equivalent in pipes. This crossing is currently expected to allow the capacity of flow from the railway pedestrian underpass, LN1 break out flow on the eastern up steam side which moves down through Egan Street as well as the flow in LN1. This was also recommended in a report commissioned by Gordonstone Coal Management PTY LTD and prepared by Blain Johnson PTY LTD in 1991.

Figure 10: Gregory Highway crossing flow structure

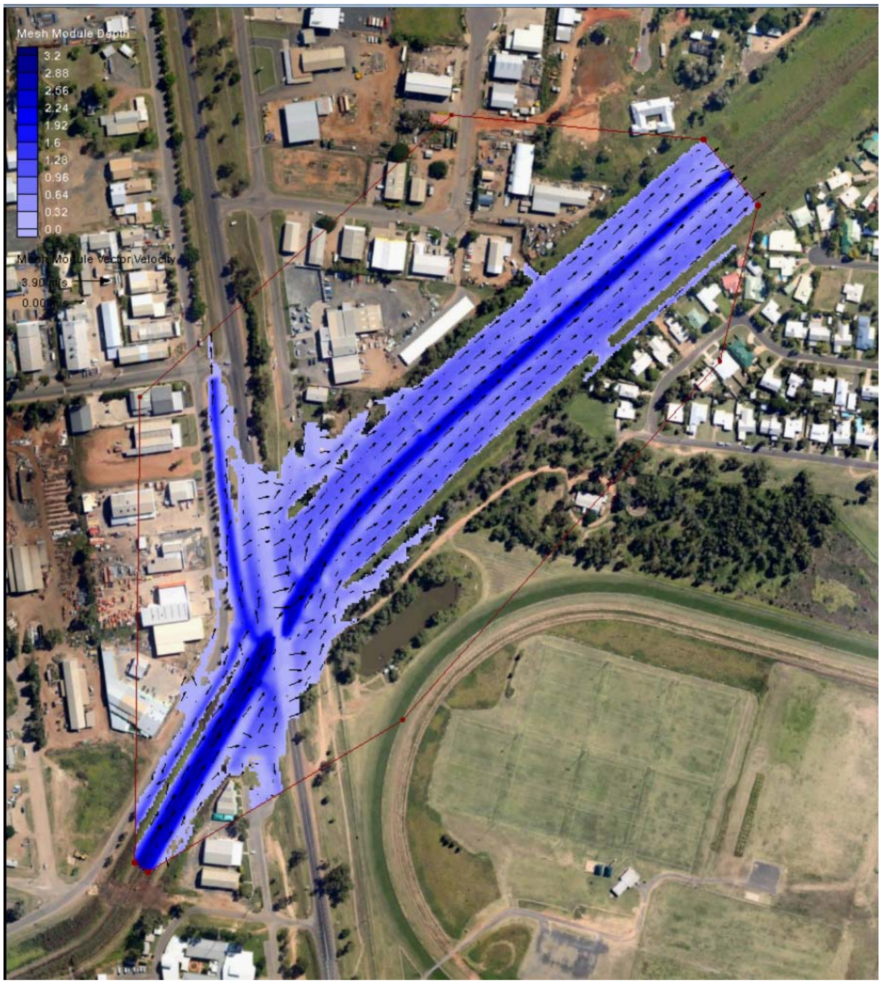

Figure 11: LN1 modelled current flow

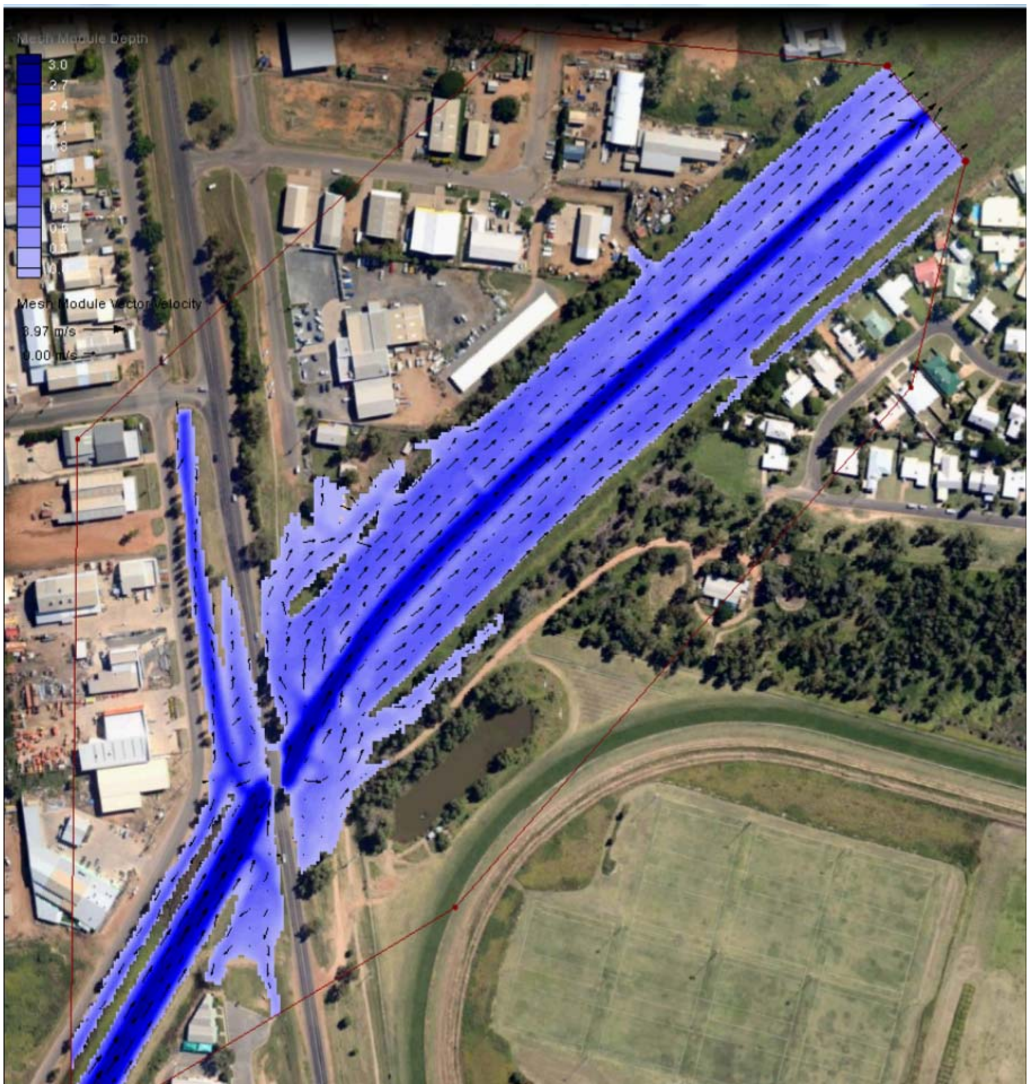

Figure 12: LN1 with recommended flow capacity increase

6.3.4 WIDENING OF LN1 DRAIN FROM HOGANS ROAD TO GREGORY HIGHWAY

The capacity of this section of LN1 is restricting the flow from Hogans Road through to the Gregory Highway. The developments on both sides of this section of drain have added to the problem by accumulating more water and restricting out of bank flows. The recommendation is to widen the drain on the Munro Road side by 15m. This would require the estimated removal of 30 000 m^3 of material and increase the capacity of this section of drain by 33%. It is recommended that the elevation of the northern embankment be set at the elevation of Munro Road. All previously commissioned reports reviewed, did not deal with the flows leaving LN1 upstream at Tyson Road and moving to the east of LN1. This recommendation must be implemented with the Gregory Highway improvement.

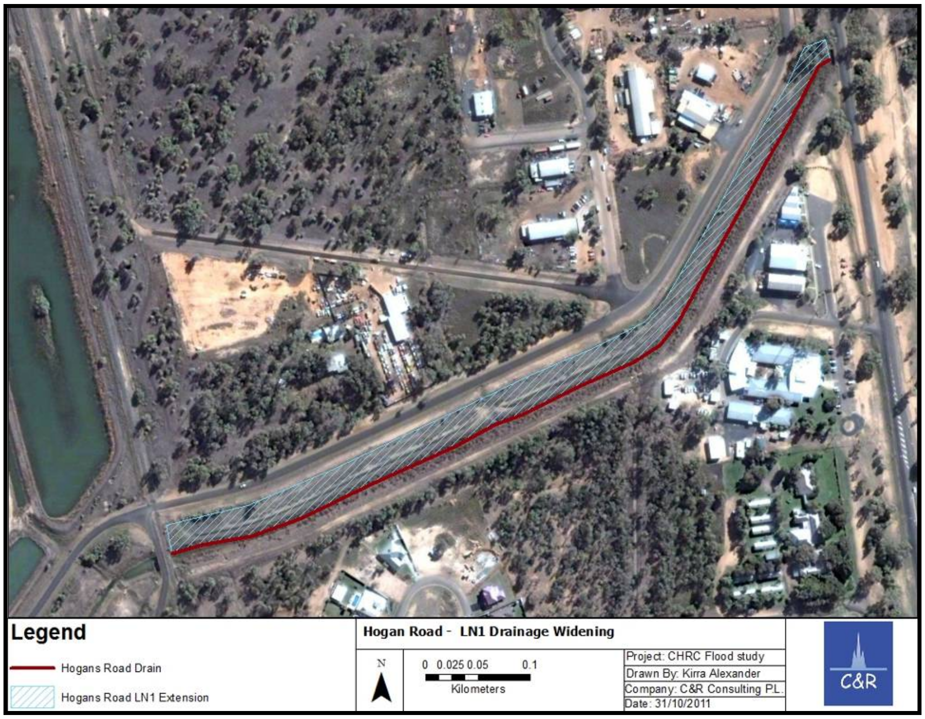

Figure 13: LN1 along Hogans Road. Recommended modifications

The blue hashed area marked in Figure 138 indicates where the bank would be widened to, and this widening has been modelled to determine the effects on water through-put along this section of the drain. Figure 9 And Figure 1510 show the modelled effects of half an hour's flow at a rate of 100 m^3/sec, both before channel widening and after channel widening.

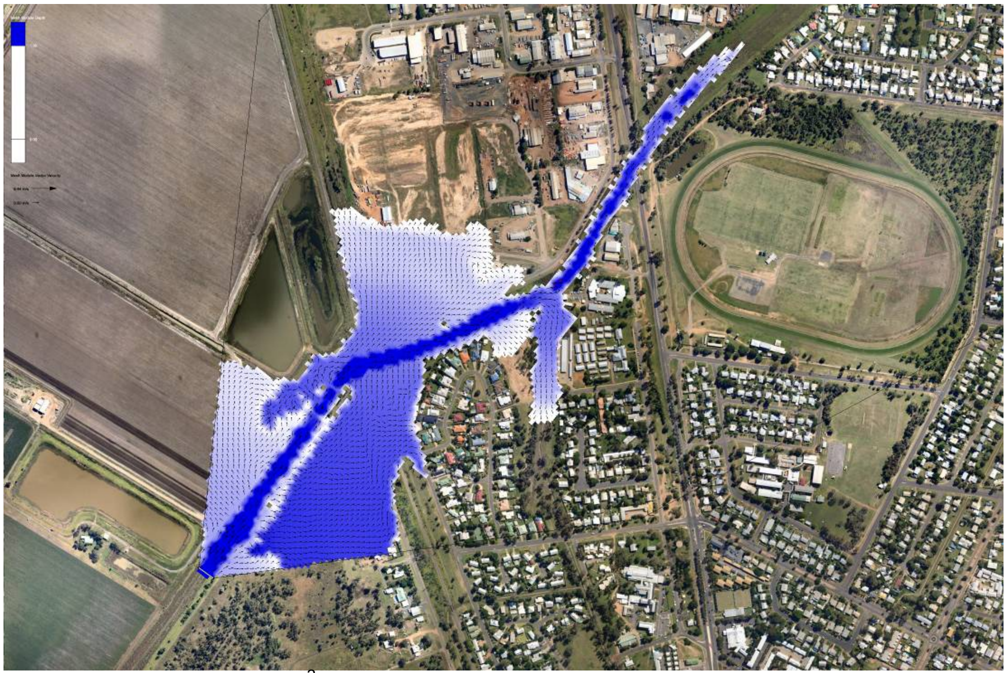

Figure 14: Modelled 100 m^3/sec flow along LN1 drain

Figure 15: Modelled 100 m^3/sec flow along widened LN1 drain

These figures show that the amount of flooding is drastically reduced in the Cameron Rd industrial estate to the north of Munro Rd. Similarly the extra water storage and flow within the drainage channel reduces the out-of-bank flow to the south through the parkland leading toward Egan St.

6.3.5 SECTION REMOVAL OF ELEVATED BANKS IN LN1 DRAIN ALONG MOFFAT ROAD

The recommendation is for the Drain embankments to be returned to an elevation taken from natural ground surface 50m south of the southern drain bank for each section; one section 200m long in LN1 and two sections 200m long in the LN1/2, all in the northern and southern embankments along Moffat Road. The three sections to be removed are described as follows:

The first section starting 900m east from the Gregory highway in LN1/2

The second section starting 1350 m east from the Gregory Highway in LN1/2

The third section starting 1950m east from the Gregory highway in LN1

The approximate positions of the section recommended for removal are marked in pink, blue and green on figure 11.

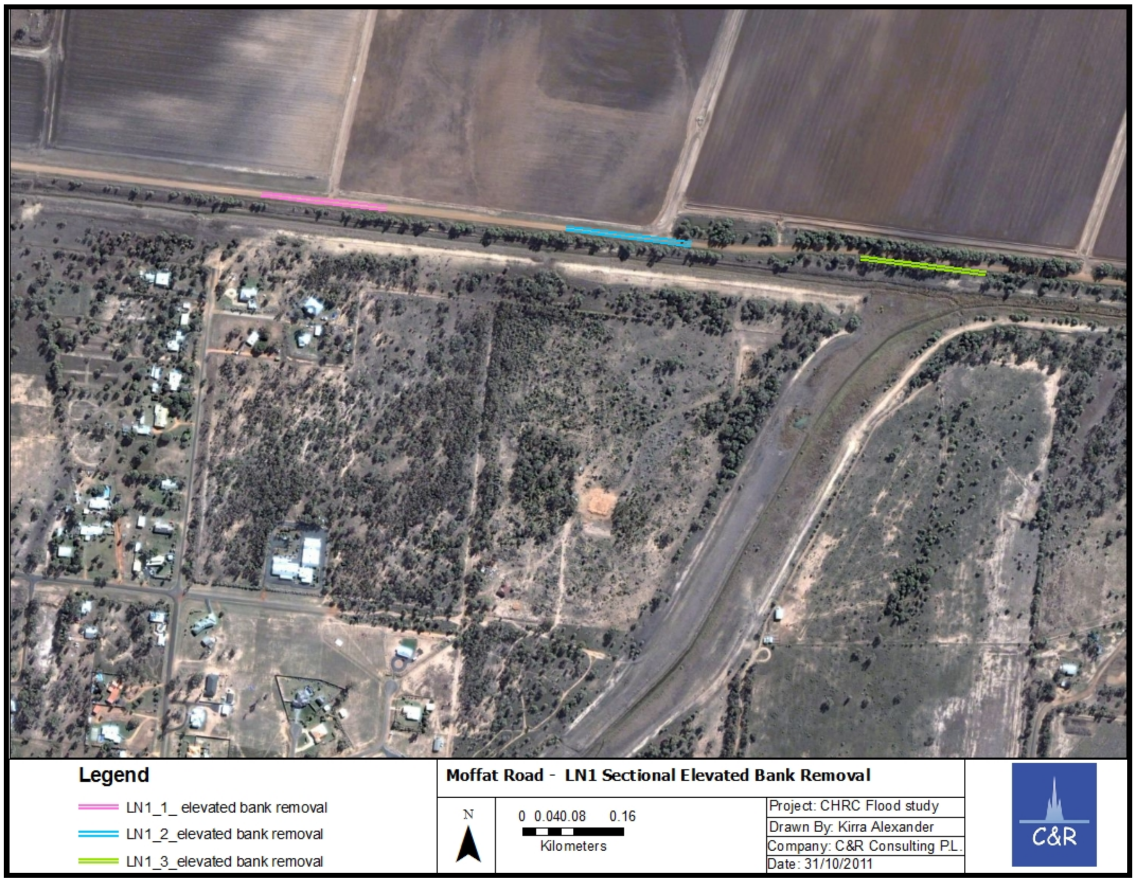

Figure 16: Elevated Banks in LN1 Drain along Moffat Road

It is recommended that the placement of these openings be worked through with the landholders immediately downstream. Other changes downstream from the openings may be required to allow the flows to return to their natural flow paths. The recommendation for these openings is to allow large flooding events to return to their natural flow paths and reduce the pooling while at the same time allowing low to medium flows to continue within the constructed drains

6.3.6 INCREASED FLOW UNDER CLERMONT STREET

The current drainage under Clermont Street is 1 x 600mm pipe. It is recommended that it be replacing with 3 x 750mm pipes which will match the flow passing through from New Street. By matching the flow volumes, the ability to move through areas of town during large events will become easier and less restricting.

6.3.7 INCREASED STRUCTURE UNDER RAIL LINE AT OLD SHEEP YARD PLACE

During large out of bank flows, all water from south Emerald tries to make it through the structure under the rail line at Old Sheep Yard Place. Modelling has also indicated that this area is very prone to flooding if any local rain were to fall when the River has the south Emerald drain backed up. It is recommended that two steps be taken to reduce the impact of flooding and allow for drainage of water in this area

1. Increase the current flow under the rail line to 3 x 1200mm x 900mm box culverts. The current structure is (2) x 1200 x 600 and (1) x 900 x 300 box culverts.

2. Open the drain parallel to Powel Street to direct water from Powell and Roberts Street to the increased rail line structure. There is 300mm of fall from the corner of Powel and Roberts to the base of the current flow structures under the rail line.

6.3.8 INCREASED FLOW STRUCTURE FOR WESTERN RAIL LINE

The current flow under the rail line is restricted to 4 x 1.5m and 6 x 1.2m pipes. The flows through this area during the 2010/11 event were over 300m^3/sec and over 160m^3/sec in 2008 event. The restriction of flow is causing water to pool and back up on the top side of the rail line until eventually overtopping the line. There has been evidence during the last two events that the over topping of the rail line and resulting breach of the structure has caused a surge of water downstream. The pooling of flows on the western side of Emerald has reduced the timing of the peak height of the last two events by 10 hours when compared to the height at the town bridge.

The recommendation for this area is to replace the current pipes with 14 box culverts measuring 3x2.5 m or a similar structure with the same flow capacity. These culverts have been modelled at a flow of 150 m3/sec for 3 hours and Figure 17 and Figure 18 show the difference in flow extent between the modelled current culverts and the recommended culverts.

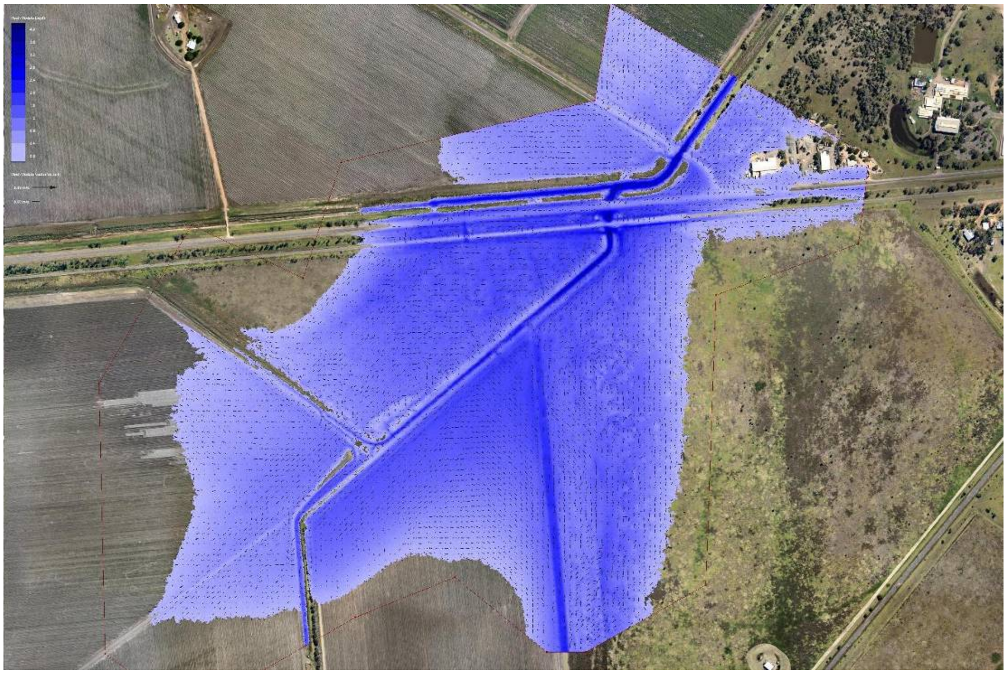

Figure 17: Current Culvert modelled flow extent

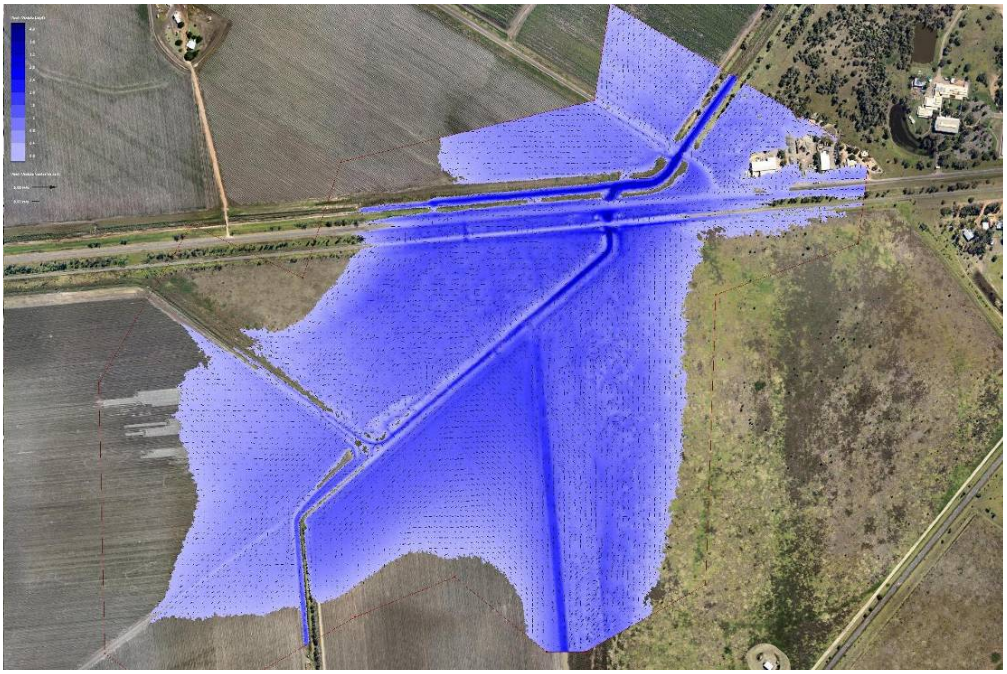

Figure 18: Recommended Culverts modelled flow extent

Figure 19: Structure under the western rail line

6.3.9 THE REMOVAL OF ELEVATED BANKS ON LNI CAPRICORN HWY

It is recommended that the elevated earth banks on the north side of the Capricorn highway on LN1 be removed to achieve the same elevation as the LN1box culvert bridge on Tyson Road from the junction of the Capricorn HWY and Tyson Rd west for 350m. This will allow for the large out of river flows to move through this area without being redirected into the town area. The removal of these levee banks was also modelled and the flow extent of this model is show in Figure 20.

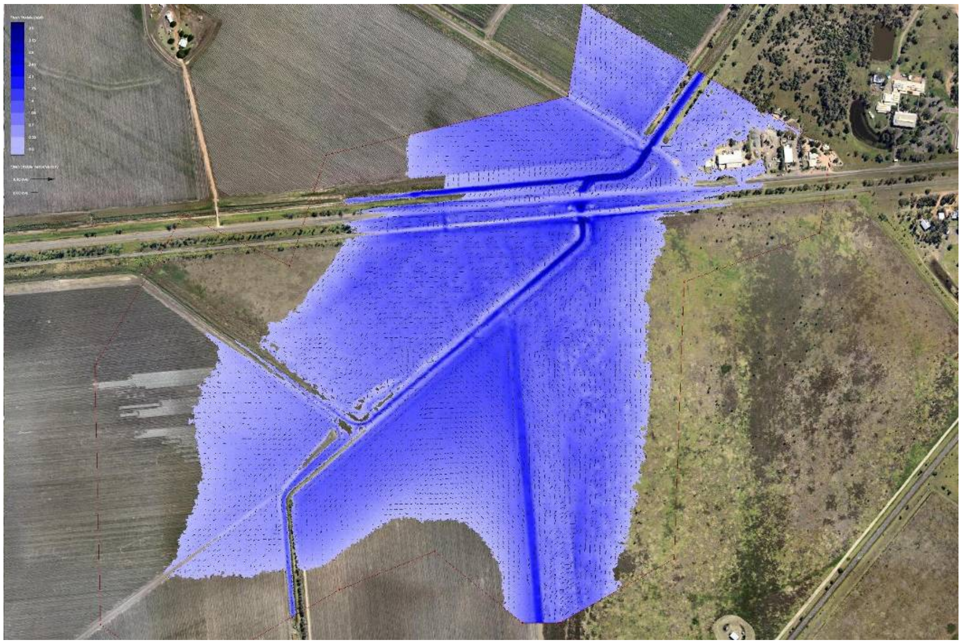

Figure 20: No levee banks with Recommended Culverts modelled flow extent

Figure 21: Elevated earth banks on Capricorn Highway LN1

6.4 RESILIENCE

The long term goal for the town of Emerald is resilience to out of bank flooding of the Nogoa River. Due to the nature of the catchment and climate dynamics of the seasonally arid tropics, the Nogoa River will always be a system that floods; however the township of Emerald can be saved from the worst effects of this flooding by designing and planning for life on a floodplain. To achieve this state of being the following actions would be recommended;

- Early warning with the use of accurate landholder information and a rainfall runoff model to minimise impact.
- Structural improvements which give access across town during an event and quickly thereafter.
  - Access will give people more time to protect their property and enable evacuation in be carry out in a safe manner. The areas of recommended structural changes are New Street. Gregory Highway across LN1. Clermont Street in front of pedestrian rail line underpass. The increased flow path under the rail line at Old Sheep Yard Place and increased drainage from the corner of Powel and Roberts Street.
    - (i) The improvements to New Street would allow access to south west Emerald during large events that wasn't possible in 2008 and 2010/11.
    - (ii) The increase in flow structures in the Gregory Hwy at LM1 drain will increase the time available for evacuation to the north by 12 hours and greatly reduce the risk of the road being washed out as in 2010/11.
    - (iii) The flow structures under Clermont Street downstream of the pedestrian rail line underpass will greatly improve access through this area during and after events.
    - (iv) The increased flow path under the rail line at Old Sheep Yard Place and drainage from the corner of Powel and Roberts Street will reduce the pooling of water in this area and allow more time for protection of property and evacuation.

- Structural improvements that maximise the flow of water through the flood plain and limiting the impact on the town. .
- The protection of flow path corridors across the town planning and flood plain areas to minimize damage.

6.4.1 FLOOD HEIGHT MARKERS

To assist in another flooding event, flood height markers from the 2010 event are recommended as a reference point for people to understand the possible risk and impact to their property. In consultation with household owners, 8-12 sites are suggested in less sensitive areas around town. It could be extended to the Vince Lester Bridge being home to a number of flood levels from the 1918, 1950, 1956, 2008 and 2010/11 flood heights. They may represent something like the image below of flood markers in South Lismore.

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Figure 22: Flood Marker Example

REFERENCES

Blain Johnson Pty Ltd (**February** (1991): Investigation of Flood Problems in Emerald. Report prepared for Gordonstone Coal Management Pty Ltd..

C&R Consulting (2010): Average Recurrence Intervals – A Statistical Review. Report Presented to the Queensland University of Technology.

Department of Natural Resources (July 1995): Nogoa River Flood Plain Study: Runoff Routing Model Calibration Report. Report prepared for Nogoa River Flood Plain Steering Committee.

Kinhill Pty Ltd (1993): Emerald Stormwater Drainage System (DRAFT). Draft Report prepared for Emerald Shire Council.

Kinhill Pty Ltd (2000): Emerald Town Flood Study - Hydraulic modelling Nogoa River Sensitivity Report. Report prepared for Emerald Shire Council.

APPENDIX 1

C&R Consulting Research Paper – ARI Statistical

APPENDIX 2

Flow Structures around Town

Location	Longitude	Latitude	Structure	Measurements	Image
Gregory HW LM1			Box culverts	2 openings x 2.0m x 2.0m 3 openings x 2.0m x 2.1m	No image
LN1 West Bank (Hogans Rd)	55 K 0617305	UTM 7399473	Box culverts	4 openings x 3m x 2.5m	
LN1 Drain Walkway Gregory Highway	55K 0617305	UTM 7399473			
Western Rail line Cap Hwy	55K 0615954	UTM 7397667	Pipes	4 pipes x 1.5m 6 pipes x 1.2m	

Capricorn Highway west	55K		Pipes	4 pipes × 1.5m	
Western Rail Line Cap Hwy	55 K 0615628	UTM 7397630	bridge	Bridge 7m × 1.8m	
Capricorn Highway (west) far bridge	55K 0534439	UTM 7560738		1 pipe × 600mm	
LN1 Drain on Braeside Road	55 K 0618540	UTM 7400519		3 × 2.4m × 1.5m	

Hogans Road Rail Bridge	55K 0617502	UTM 7398929		13m × 1m	
Tyson Rd LN1	55K 0616110	UTM 7397806	Box culvert	3 openings × 1.8m × 3.3m	
Centre of town rail line walk through	55K 0618087	UTM 7397855	Box culvert	Opening 1.7m × 1.8m	
Centre of town HWY (directly opposite above)	55K 0618089	UTM 7397854	pipe	600 mm	

CLIENT: Central Highland Regional Council
PROJECT: Flood Management Analysis
REPORT: Section B – Fairbairn Dam through Emerald
DATE: December 2011

Long Street Dip	55K 0618290	UTM 7397255		330mm		
Rail Line (street Corner)	55K 0617578	UTM 7397798		(2) x 1200 x 600 (1) x 900 x 300		
Rail Line (2)	55K0616938	UTM 7397740		(2) x 450 x 300		
Rail Line (3)	55K 0616668	UTM 7397714		900 x 300		

Capricorn Highway (TAFE)	55K 0616665	UTM 7397749		(2) x 1200 x 600		
Capricorn Highway (Landmark)	55K0616933	UTM 7397784		(3) x 1200 x 450		
Capricorn (Rail line crossing)	55K0617785	UTM 7398068		(2) x 1200 x 450		
Vince Lester Bridge	55K 0619123	UTM 7397247				

Nogoa Rail crossing	55K 0619117	UTM 7397247		Refer to attachment	

Cross sections in LMI every 4m stating from north bank.

163 0 .45 1.15 1.85 2.9 3.5 3.6 4 4 3.3 3.2 3.2 3.2 2.7 2.25 o.9 0
162 1.35 1.3 1.8 2.8 3.3 3.35 4 4 3.2 2.75 1.88 1.85 1.35 1.08