2 Introduction

85. The Grantham Floods Commission of Inquiry (the GFCOI) was established under the *Commissions of Inquiry Act 1950* to make full and careful inquiry into the flooding of Lockyer Creek between Helidon and Grantham on 10th January 2011.

86. I have been engaged by the GFCOI to provide an expert hydrology opinion in relation to:

- the likely chronology of the flooding between Helidon and Grantham on 10th January 2011; and
- the possible factors that may have altered, contributed, caused or materially impacted on the flooding on 10th January 2011 with specific reference to any natural or man-made features of the landscape, and in particular the Grantham Quarry.

87. In forming my opinion I have been requested to consider the matters set out in four letters of instruction contained in Appendix E.

3 Qualifications and Experience

88. I am a Chartered Professional Engineer practising in the area of Water Engineering, with over 35 years experience. I graduated from the University of Queensland with a Bachelor of Engineering (Civil, Honours) in 1978 and later completed a Doctor of Philosophy in 1991 on river hydraulics. I am an Honorary Fellow of Engineers Australia and a Registered Professional Engineer of Queensland. I am also the Director of Water Solutions Pty Ltd, a consulting company that I established in 1995. A copy of my Curriculum Vitae is included in Appendix D.

4 Conflicts of Interest

89. It also may be of relevance to some for me to note that I was not directly involved with the previous Queensland Floods Commission of Inquiry (QFCOI). Neither do I have any conflicts of interest with any stakeholders or interested parties. The opinions that I present in this report are on the basis of an objective independent assessment with the benefit of hindsight, new information and access to the work of others.

5 Structure of the Report

90. I have structured my report as follows:

- Index;
- Glossary of Terms;
- Nomenclature;
- Section 1 – a summary of my investigations and conclusions reached;
- Section 2 – the introduction to my report;
- Sections 3 and 4 – statements about my qualifications and conflicts of interest;
- Section 6 – key information that I have referenced;
- Section 7 – what caused the flooding of the Lockyer Creek between Helidon and Grantham on 10th January 2011 and its likely frequency of occurrence at Grantham;
- Section 8 – the flood model that I prepared for the purposes of my review (the GFCOI model). The main differences between my computer simulations and those by others are highlighted;
- Section 9 – my assessment of the most likely chronology of the 2011 flood;
- Section 10 – quantifying the likely effect of the Grantham Quarry on the Grantham flooding;
- Section 11 – quantifying the likely effect of the railway embankment on the Grantham flooding;
- Section 12 – testing the performance of the GFCOI model;
- Section 13 – comparison between eye-witness accounts and computer model simulation;
- Section 14 – flood evacuation routes and the likely effect of the Grantham Quarry on these;
- Appendix A – references to material that I have referred to;
- Appendix B – more details of supporting technical analyses that I have undertaken as part of my investigations;
- Appendix C – a selection of enlarged figures for ease of reference;
- Appendix D – my Curriculum Vitae; and
- Appendix E – the letters of instruction issued to me by the GFCOI.

6 Key Information

6.1 Overview

91. My investigations have drawn on information sourced from the following:

- eye-witness accounts of the flooding between Helidon and Grantham;
- an inspection of Grantham, the Grantham Quarry, the floodplain and surrounds on 1 June 2015;
- DNRM stream gauging stations historical height and flow records;
- hydraulic modelling data for the study area and supporting reports prepared by SKM (now Jacobs) for Lockyer Valley Regional Council (LVRC 2014);
- hydraulic modelling data for the study area and supporting reports prepared by SKM (now Jacobs) for the Queensland Flood Commission of Inquiry (Jordan 2011);
- additional peak flood level data not considered by previous flood investigations; and
- expert geotechnical advice from Mr Starr (Golder Associates, 2015) in relation to soil horizons around the Grantham Quarry.

92. I have also reviewed the report prepared by DHI for Nationwide News Pty Ltd entitled *Grantham and Wagner Quarry, Review of Flood Impact 10th January 2011 Flood Event, February 2015* (Szylkarski 2015).

93. A complete list of materials that I have relied upon is provided in Appendix A.

6.2 Eye-witness Accounts and Statements

94. Eye-witness accounts have provided me with significant insight and have directly contributed to the substance of the outcome of this investigation. In particular eye-witness accounts have:

- assisted me with identifying the most likely circumstances of influence to the sequence and action of the 2011 Floods;
- allowed me to corroborate the efficacy of computer simulation outcomes; and
- assisted with my interpretation of computer simulation outcomes.

95. It is of significance to note that I have found that many of the eye-witness accounts are consistent in their description of the characteristics of the 2011 Floods. I have found that they generally provide a consistent account against which I have been able to achieve a satisfactory computer simulation. The eye-witness accounts have provided vital information with which to better define the critical time of arrival of the flooding of Grantham. A selection of the statements that I have specifically made reference to in my report are as follows:

- Arndt, Frances Ann, statements dated 29 January 2011 and 1 July 2015
- Besley, Graham Francis, statement dated 2 July 2015
- Besley, Helen, statement dated 2 July 2015
- Cork, Richard, statement dated 2 July 2015

- Lack, Wayne Douglas, statement dated 7 July 2015
- Mallon, Neville Lester, statement dated 1 July 2015
- McIntosh, Anthony, statement dated 1 July 2015
- Richardson, Lance William, statement dated 1 July 2015
- Sippel, Jonathan, statement dated 1 July 2015
- Steffens, Troy Brenden, statement dated 1 July 2015
- Zischke, Gavin Noel, statement dated 9 July 2015

96. I also have referred to:

- the following material concerning Mr Bruce Marshall:
  - an excerpt from the Coroner's report regarding the inquest into the January 2011 flood deaths. That excerpt concerns the death of Mr Bruce Marshall; and
  - transcripts from 000 calls made by Mr Marshall on 10 January 2011.
- the following additional material concerning Mrs Besley:
  - transcript of 000 call referred to at paragraph 21 of Mrs Besley's witness statement dated 2 July 2015 (separate from her statement); and
  - excerpt of transcript of hearing before the GFCOI on 22 July 2015 (a copy is attached to Letter of Instructions No 2 provided at Appendix E);
- excerpt of transcript of hearing before the GFCOI on 21 July 2015 regarding the examination of Mr Sippel (a copy is attached to Letter on Instructions #4 provided as Appendix E); and
- iPhone photographs taken on 10th January 2011 and supplied by Mr McIntosh (separate from his statement).

97. I have also reviewed many other statements during the course of investigations. Collectively these statements provide a fairly complete description of the event as it unfolded for the people of Grantham. A full list of statements reviewed in this investigation is included in Appendix A.2 and A.3.

<u>Key Characteristics</u>

98. A selection of key characteristics of the 2011 Flood that have proved useful to me in developing an understanding of the circumstances surrounding the flooding at Grantham have been extracted from the above material and are summarised as follows:

- At between 3:08pm and 3:18pm, a surge of water was observed in Lockyer Creek adjacent to Klucks Road, approximately 700m upstream from the Quarry (Mr McIntosh's statement).
- At about 3:30pm (statement time of 3:00pm adjusted to match the time of the 000 call) Mr and Mrs Besley were standing on a concrete slab on the north-eastern bank of Lockyer Creek adjacent to the Grantham Quarry and were observing water levels in the creek (Mr and Mrs Besley's statements). In the excerpt of the 2015 transcript, Mrs Besley referred to observing water flowing over the location of the haul road. In my opinion, it is likely that flood levels in Lockyer Creek at this location had already risen and inundated the road by this time. Shortly after this time the creek levels began to rapidly rise. The Besleys tried to escape by car but were overcome by water. The initial breakout of floodwater at this location had intercepted

the Besleys in their car by 3:48pm (000 transcript). They were subsequently washed from their vehicle at close to 3:59pm (000 transcript).

- At 3:56pm, the Gatton-Helidon Road through Eastern and Western Grantham had not been inundated, but the bridge crossing of Sandy Creek had been overtopped by this time (Mr Steffens' statement).
- At 3:58pm, the Gatton-Helidon Road near the intersection with Sorrensen Street was close to being inundated (Mr Steffens' statement).
- Normally Grantham is flooded by water backing up Sandy Creek (Mr Lack's and Mr Richardson's statements).
- Flood inundation of Western Grantham was initially by fast flowing rapidly rising floodwater emanating from Lockyer Creek to the southwest (Mrs Arndt's statement, prior to 4:07pm). Mrs Arndt described it as a big extended roll of water.
- Mobile telephone records put 4:07pm as close to the time when significant floodwaters overtopped the Gatton-Helidon Road, at about midway between Sorrensen and Citrus Streets (Mrs Arndt's statement).
- Mobile telephone records put 4:09pm as close to the time when flood depth and velocity of floodwater over the Gatton-Helidon Road, at close to house number 1348, reached a point sufficient to compromise the stability of a Rural Fire Brigade truck (Mrs Arndt's statement).
- At 4:10pm, the Gatton-Helidon Road near the intersection of Dorrs Road had become flooded (Mr Steffens' statement).
- A 000 transcript puts 4:10pm as close to the time when floodwater at 1420 Gatton-Helidon Road (Mr Marshall's residence) reached the floor level of a low set timber dwelling. 8 minutes later the depth of water within the dwelling was shoulder deep.
- Flooding of Central and Eastern Grantham initially occurred from Sandy Creek (between after 3:00pm and before 4:14pm) but this was soon followed by fast flowing rapidly rising floodwater from the west at 4:14pm (Mr Lack and Mr Richardson's statements).
- The initial inundation of Grantham was followed by a subsequent surge of flow. The time of this second surge at adjacent to 26 Anzac Avenue was about 4:23pm (Mr Lack's statement) where a significant rise appeared to have occurred in *a matter of seconds* (Mr Lack's description). Although this subsequent additional flood flow headed through Eastern Grantham, its source was not witnessed. This is also consistent with a report of a significant rise at the Grantham Hotel over the period between about 4:15pm and 4:53pm (Mr Richardson's statement).
- Inundation of the floodplain immediately upstream of the quarry continued after the passing of the peak of the flood. A photograph supplied by Mr McIntosh at 4:41pm (IMG_0236.jpg from an iPhone) shows continuous inundation of the entire floodplain from his residence at Klucks Road, to the upstream of the quarry in the north-east and to the Gatton-Helidon Road in the north.

<u>Survey Data</u>

99. I have also reviewed additional survey data of peak flood heights that has assisted me with corroboration of simulation outcomes from the GFCOI model from the following material:

- Rickuss, Ian, Flood Level Survey Plan TM153 FL 002A
- Cork, Richard, statement dated 3 June 2015.

6.3 Inspection of the Grantham Quarry, Grantham and Surrounds

100. On 1 June 2015, I inspected the quarry, Grantham and surrounds. The purpose of the inspection was two-fold:

- gain general familiarity with the area; and
- establish the scope of investigations to be undertaken by Mr Starr (Golder Associates) to meet with the needs of the subject hydrology investigations.

101. The inspection commenced at the quarry in the company of Mr Starr and a representative escorted us from Boral. The role of the Boral representative was to ensure that Boral Workplace Health and Safety requirements were adhered to for the benefit of all parties during the conduct of our inspection.

102. The full area of the quarry pit was circumnavigated during the course of the inspection. I paid specific attention to the creek channel and adjacent overbank areas, the western side of the pit and existing levee banks, the location where the Main Breach occurred, the broken power pole on the Western Levee and the state of the inside pit walls including a large scoured flow path in its south-eastern corner. A track record of my inspection is shown in Figure 6.1.

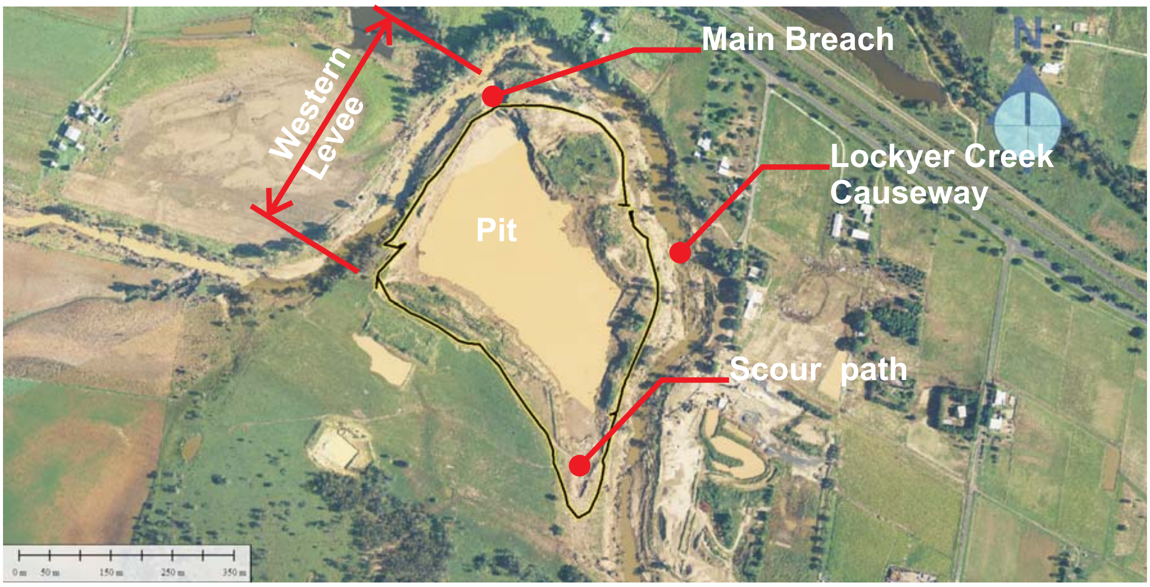


Figure 6.1 – Track of Field Inspection at Quarry

103. My inspection of Grantham and surrounds then followed. It included a detailed drive around the streets of Grantham from the fuel station to Sorrensen Street, both sides of the rail embankment, Sandy Creek in between the railway and Gatton-Helidon Road, the entire length of the Gatton-Helidon Road, and the road following Lockyer Creek downstream from Helidon to Carpendale via Kapernick's Bridge.

6.4 DNRM Stream Gauging Stations

104. DNRM operate a number of stream gauging stations in the Lockyer Valley. The purpose of the stations is to automatically keep time-stamped water level records from which estimates of flow rates can be derived. There are 3 stations in reasonably close proximity to Grantham that are of relevance to my investigations:

- 143203C – Helidon
- 143233A – Flagstone Creek
- 143213C – Ma Ma Creek

105. There are a number of other stations in the vicinity, but I did not consider these to be of direct relevance to my investigations. For example, DNRM operates a gauging station on Tenthill Creek, but this is not of direct interest to me as the creek enters Lockyer Creek well downstream from Grantham sufficiently far afield to not be of consequence to my investigations.

106. I also note that there are no DNRM stream gauging stations on Sandy and Monkey Waterholes Creeks. As a consequence, my investigations have instead included estimates of flows from these creeks derived using records of rainfall.

107. Estimates of rate of water flow in the creeks at any point in time can be derived from the water level records from the DNRM stream gauging stations by the use of a *rating table* or a *rating curve*. These tables / graphs are a form of ready reckoner that enable flow rate to be determined from the water level.

108. Flow rates are usually expressed in terms of cubic meters per second (m^3/s), where one cubic metre is 1,000 litres. Another term for a m^3/s is a *cumec*. Water level is usually measured relative to the approximate bed level (called Gauge Zero) and is referred to as Gauge Level or GL.

109. Rating tables are best determined by physical measurement of flow velocity in conjunction with the associated water level and a surveyed cross-section. Unfortunately making direct flow measurement requires timely and safe access to the site that is often difficult to achieve during short duration rare flood events. For example, the highest flow rate measured to-date at the Helidon station is $108m^3/s$ recorded in 1988 (3.4 mGL). This flow rate is very small relative to DNRM's estimated flow rate of $4{,}100m^3/s$ (11.38mGL) at Helidon for the peak of the 2011 Floods.

110. Extension of a rating table above the highest measured flow may be undertaken using a variety of approaches, for example graphical extrapolation of the rating curve (i.e. by eye) or by a range of mathematical calculations. However, the type of calculation applied is often relatively simplistic and not necessarily very accurate.

111. A detailed review of the Helidon Gauging Station flow rating is presented in Appendix B.2.

112. Data from DNRM automatic water level recording stations is freely available via:

- www.dnrm.qld.gov.au/water/water-monitoring-and-data/portal

6.5 Computer Models and Associated Data

General

113. Computer models of real life processes are used to help engineers better understand and manage complex processes. Flood assessment is a complex process. However, the complexity of the flooding process can be objectively broken down into a set of individual processes that can be defined and quantified in a logical and scientific manner.

114. However, modelling typically requires reality to be simplified based on the theory underpinning the model in question. This means that model simulations will rarely exactly match reality in all respects. In other words, computer simulations provide a representation, or approximation, of reality.

115. Another factor that also affects how accurately models represent reality is the availability of suitable information with which to construct the model. This information comprises:

- data that can be reasonably well measured spatially (e.g. topography from LIDAR Survey aerial mapping);
- data that can be measured at discrete locations (e.g. rainfall depths at rainfall gauges); and
- data used to calibrate the various mathematical formulae used in the models (e.g. the effective roughness that water sees when it flows over the ground needs to be described indirectly by a parameter that makes sense to the formulae, but is not easily directly measurable).

116. When interpreting the outcomes of simulation models I consider it important to always keep in mind the limitations of the computer model being used. One key item of relevance to my investigations is that the simulated water levels will always be presented in simulation outcomes as a relatively smooth surface. This is because the simulated water surface is in effect the average over the modelling grid cell. It will not pick up the higher resolution details such as, for example, sheds with plan dimensions less than 10m. Also, the flow of fast moving floodwater usually produces a relatively choppy surface, sometimes with small wavelets and much turbulence. Hydraulic model simulations will present this same surface smoothed out. It will still have the same average water surface levels, but not the turbulent peaks and troughs.

117. For flooding investigations, models are used to calculate a comprehensive set of parameters, including flood flow rates, water levels, and velocities. Some flood models only provide this information along the primary flow path (or main channel); these are called one-dimensional models (1D). Others provide this information spatially distributed (that is, spread out in plan) with simulations providing “depth averaged” values (for example, in reality flow velocity varies over the depth of water, but in this case only the average velocities at plan locations are used); these are called two-dimensional models (2D). For very specialist situations full three-dimensional modelling (3D) can also be undertaken; but these are complex and expensive to setup and use.

118. Flood models can also be run in two modes: steady state or dynamic. Steady state modelling assumes that flow rates remain constant and never change; while dynamic modelling allows flow rates (and potentially other factors, such as eroding levee banks) to change over time.

119. Steady state models are good for those situations when the flooding event is relatively slow to rise and fall. In those cases the steady state flow rate used would be that for the flood peak and modelling outcomes would provide “maximum” flood event estimates. Steady state models are

not suitable for those situations where the rate of rise and fall of the flood event is relatively quick, or when the capacity of the waterway system (main channel and floodplains) to store volumes of floodwater is significant, or when details of timing are important.

120. For both steady state and dynamic flood models, it is the flow rate of the water that is the primary driving function. For example, increasing flow rate produces increasing flood levels, and velocities.

121. For dynamic modelling timing is also of key importance. Not only because this directly affects when things occur, but also because the volume of floodwater moving through a waterway system is directly affected by time. Volume is the product of flow rate and time:

- V (m^3) = Q (m^3/s) x t (s)).

122. In flood modelling the change in flow rate over time is called a “flow hydrograph”. The change in water depth over time is called a “stage hydrograph”. This hydrograph information is most often presented in a graphical form of the flow, or stage, plotted against time.

123. Two types of models are generally needed for flood investigations:

- hydrology model - produces estimates of flood flow rates, usually from rainfall data; and
- hydraulic model - produces flood levels, flow depths, velocities from the flow rates.

124. I have used dynamic 2D hydraulic (TUFLOW) and hydrology modelling (RAFTS) for my investigations.

125. I consider the 2D model that has been applied is better suited than a 3D model to my investigations into the Grantham flood.

126. The models that I used for the purposes of the GFCOI are discussed in the following sub-section.

<u>Lockyer Valley Flood Models</u>

LVRC Models

127. I have been provided with detailed flood modelling of the Lockyer Valley that has recently been undertaken by Jacobs for the Lockyer Valley Regional Council (LVRC 2014). The report provided by Jacobs on this model states that it was calibrated to peak flood level records from the 10th January 2011 flood event. The model includes representation of the Grantham Quarry, including representation of the Western Levee of the quarry.

128. The extent of the LVRC flood models also included:

- topography (1m resolution LIDAR aerial survey) for before and after the 10th January 2011 flood event (August 2010 and January 2011, respectively).
- aerial photograph mosaics for before and after the 10th January 2011 flood event (August 2010 and January 2011, respectively);
- localities and levels of surveyed maximum flood heights;
- hydrology model (RAFTS) and simulation output data for the 10th January 2011 floods; and
- hydraulic model (TUFLOW) and simulation output data for the 10th January 2011 floods.

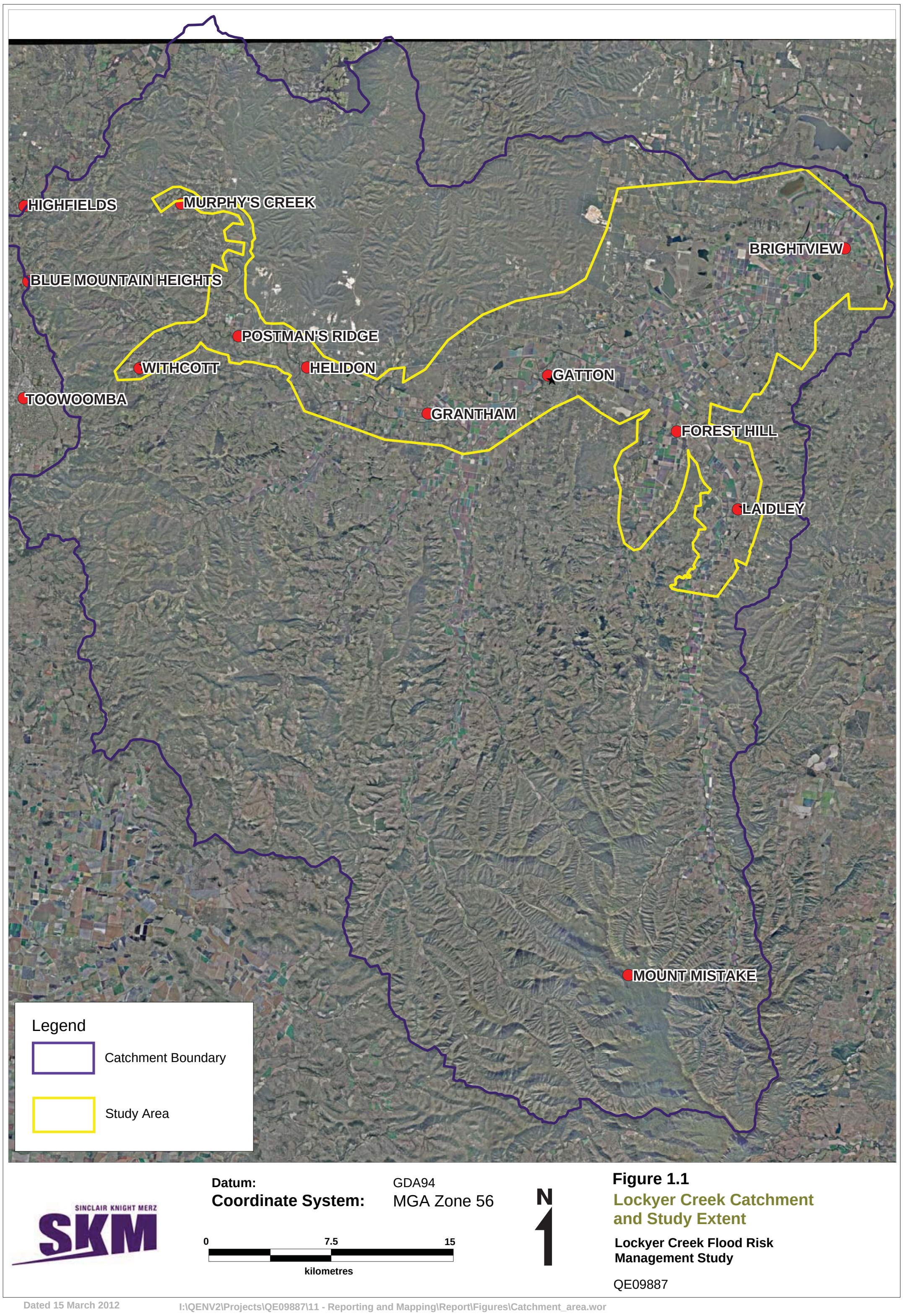


Figure 6.2 – LVRC Flood Model Extent

129. This information and models (collectively referred to as the LVRC models) were made available for these investigations, as were copies of associated Jacobs (SKM) flood study investigation reports.

130. The extent of the LVRC flood model is indicated in Figure 6.2 (Jacobs 2014) and shows Grantham located centrally.

Jordan Model

131. Jacobs also provided modelling data used by Dr Phillip Jordan in support of his submissions to the QFCOI. My review of this data confirmed my understanding that the models used by Dr Jordan were directly based on earlier versions that eventually became the final LVRC modelling package, but modified to suit the specific requirements of his work for the QFCOI.

6.6 Investigations by Geotechnical Experts

132. Mr David Starr (Golder Associates, 2015) was commissioned by the GFCOI to undertake investigations at the Grantham Quarry that included determination of the natural surface of the land separating the Quarry pit from Lockyer Creek and the floodplain flow path immediately to the south of the quarry.

133. Current investigations have considered the hypothetic scenario of what the 2011 Flood characteristics would have likely been at Grantham had there been no development of the quarry area to the south of Lockyer Creek. Information from Mr Starr's investigation has been used to establish a reasonable assessment of what the topography would have been had no development of the area been undertaken.

7 The 10th January 2011 Flood Event

7.1 Rainfall and Flood Flows

134. The flooding between Helidon and Grantham on 10th January 2011 resulted from an intense two hour storm burst that occurred on the afternoon of Monday 10th January 2011, falling on an already saturated catchment. This burst travelled in a southwest direction from the coast over Wivenhoe Dam and then across the upper catchments of the Lockyer Valley. According to Jacobs (2014), the rainfall intensified as it travelled further inland, reaching its most intense form across the Murphy's Creek, Fifteen Mile Creek and Alice Creek catchments.

135. The progress of the rainfall event was captured on Bureau of Meteorology Rainfall Radar imagery. I have presented a representative selection of images (BOM) in Figure 7.1 below. The images are from the Mount Stapleton Radar and span a period of 3 hours from 10am on 10th January 2011 to 1:00pm on the same day.

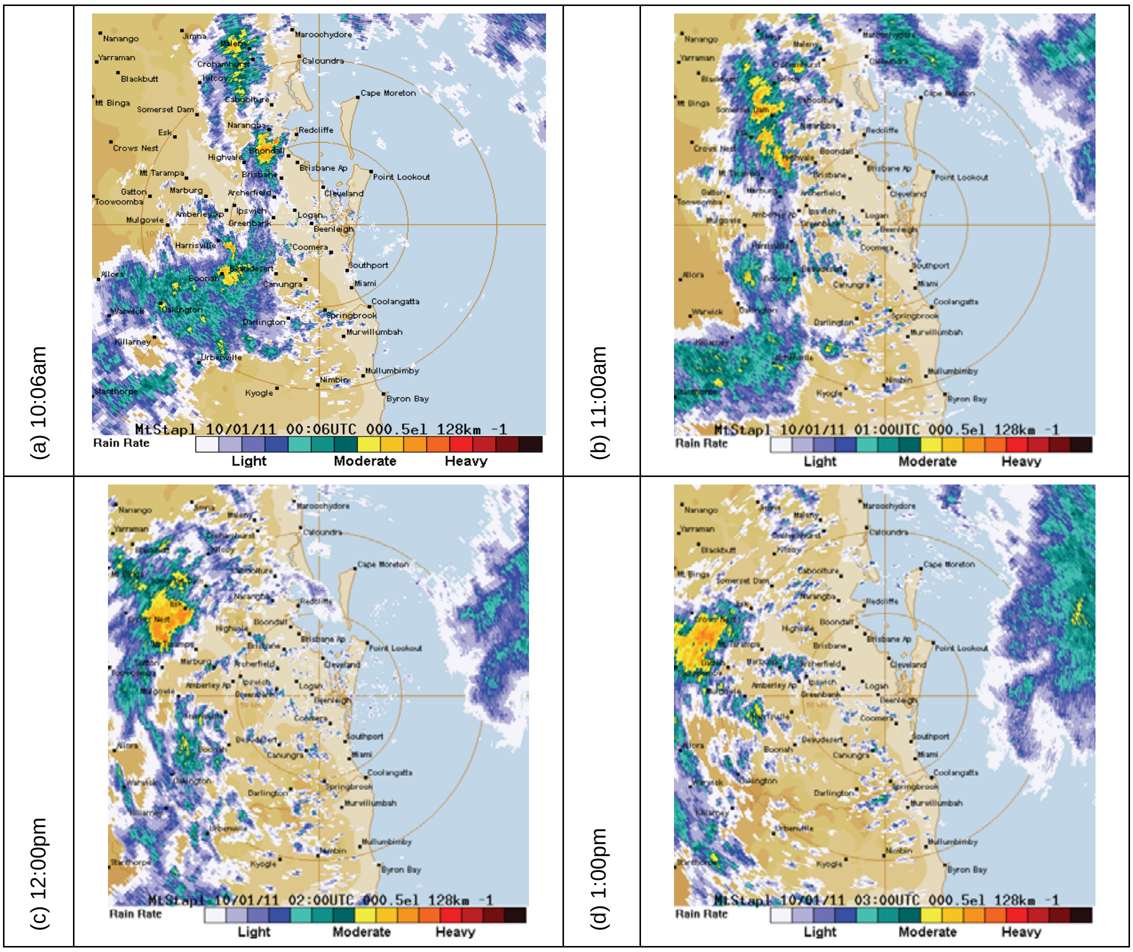


Figure 7.1 – BOM Rainfall Radar Imagery

136. Prior to the 2011 Flood Event all catchments of the Lockyer Valley and headwaters were effectively saturated on account of preceding rainfalls over December 2010 and early January 2011 (before the 10th). The Helidon Post Office (Station Number 040096) recorded daily rainfall totals of 123mm over the 7 days prior (3rd January to 9th January) to the 10th January 2011, and

503mm over the 31 days prior (6th December to 9th January). Runoff from the 10th January storm was therefore maximised as little rainfall was lost to soil moisture stores.

137. On the afternoon of Sunday 9th January (the day before the event), rain fell over most of the catchment and resulted in a relatively small flood event that flooded the lowest parts of Grantham, the second flood to inundate the town since Christmas (2010).

138. BOM (2011) investigations revealed that the 10th January 2011 floods started as two distinct areas of intense rainfall crossing the coast at about 9:00am. One was moving southwest from the Caboolture area. The other more intense area was moving west-south-west from the Redcliffe area. These two rainfall areas then merged at about 11:00am over Esk (about 40km north-east of the catchment centroid at Gatton). A single intense system then moved south-westerly towards the Lockyer catchment.

139. The shape of the weather system uniquely matched the “bowl” shape at the top of the Lockyer catchment. This resulted in the high intensity rainfall falling wholly within the upper catchment, with significantly lower amounts in the lower catchment areas (which includes Grantham). Major flooding throughout the Lockyer Valley occurred.

140. Jacobs (2014) have estimated that the rainfall intensities in the upper catchment were likely to have been about 1 in 300 AEP or larger.

141. The probability of exceedance of a storm event rainfall does not necessarily equate directly to the probability of exceedance of the resultant flood. A range of other factors also influence the probability of the resultant flood, including how wet the catchment was before the event and the pattern and timing of the rainfall event, catchment shape, channel size, and gradients.

142. A flood frequency analysis was undertaken by Jacobs (2014) to produce a relationship between peak annual flow rate and AEP at the Helidon Gauging Station. I have reproduced this graphic in Figure 7.2 (following page).

143. DNRM has estimated the peak flowrate at Helidon as 3,640m^3/s. According to Figure 7.2 this equates to an AEP of around 0.25%, or 1 in 400. Considering the broad confidence associated with an event as large as the 10th January 2011 flood I consider it is reasonable to conclude that this is consistent with the AEP of the rainfall.

144. A streamflow gauging station is located at Helidon, some 10km upstream from Grantham. The occurrence of the 2011 Flood Event recorded at Helidon was sharp. Gauging station data shows that the flood flow commenced at Helidon at just after 2:00pm and peaked at a flow depth of 8.3m in just one hour (3:10pm). I consider this to be very rapid.

145. On this basis, I consider that the inundation of Grantham on 10th January 2011 was an extreme event.

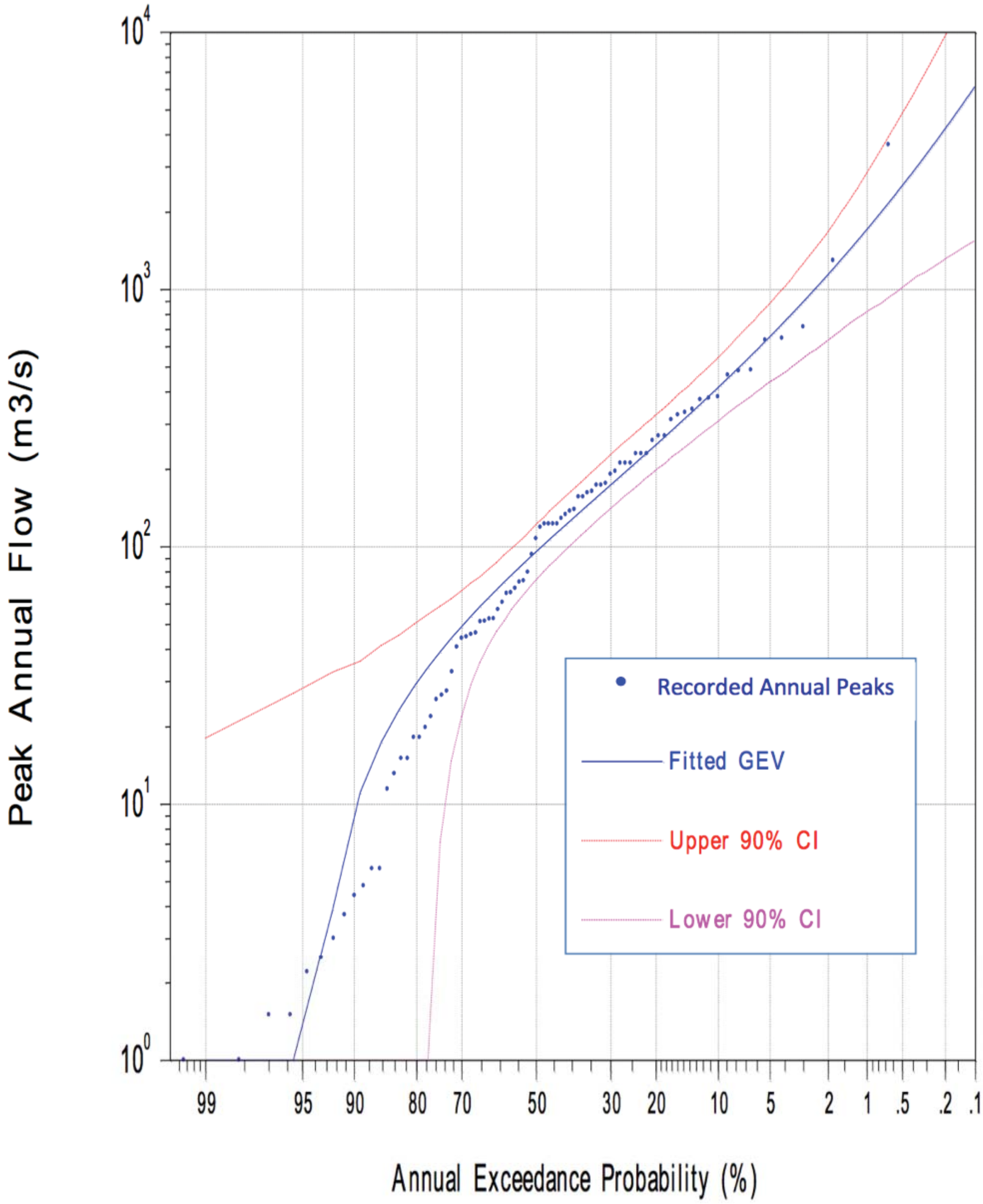


Figure 7.2 – Annual Series Frequency Analysis: Lockyer Creek at Helidon, Jacobs (2014)

7.2 Changes to the Waterway

146. A substantial debris and sediment load was generated in consequence of the 2011 Flood Event. High velocities in the tributaries of the Upper Lockyer resulted in stripping of much of the vegetation from its roots, including the felling of large trees. Jacobs (2014) estimated that the flow area of the waterway at Murphy's Creek was increased by as much as 35%.

147. Aerial photographs show that the extent of stripping and erosion reduced further downstream, see Figure 7.3. At Helidon gauging station, my comparison between pre and post flood aerial survey indicates that the cross-sectional area of the waterway to the peak level of the 2011 Flood event had increased by less than 1%. The effect on vegetation is more noticeable.

Figure 7.3 (a) – Helidon Gauge – August 2010 Aerial Photography (Station at Red Circle)

Figure 7.3 (b) – Helidon Gauge – January 2010 Aerial Photography (Station at Red Circle)

7.3 Changes to the Quarry

148. My comparison between pre and post flood aerial photography and LIDAR aerial survey shows that the quarry suffered significant damage to the levees and riverbanks during the course of the 10th January 2011 floods. In summary, I consider the key damage items of significance to my investigations were:

- significant erosion of the levee banks that separated the western side of the quarry pit from Lockyer Creek; and
- the development of a breach in the natural creek bank in the low point in the Western Levee (Main Breach).

149. The general locality of these items is indicated in Figure 7.4.

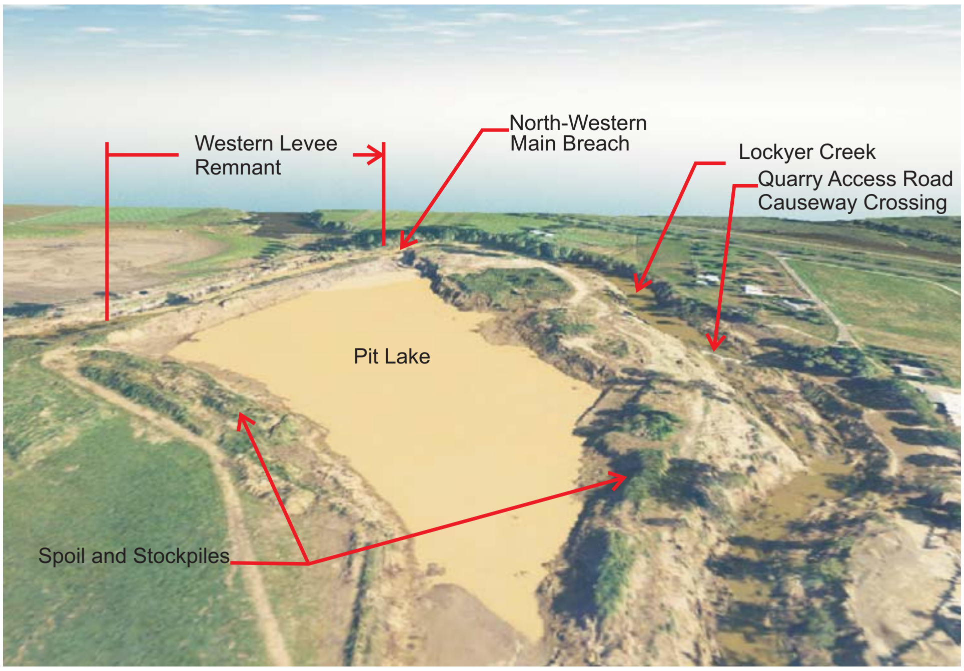


Figure 7.4 – Oblique Showing Post-flood Quarry

150. The aerial extent of this erosion damage is indicated in Figure 7.5 that presents before and after aerial images of the area. I have provided further quantification in Section 8.6.

(a) Before Flood (August 2010)

(b) After Flood (January 2011)

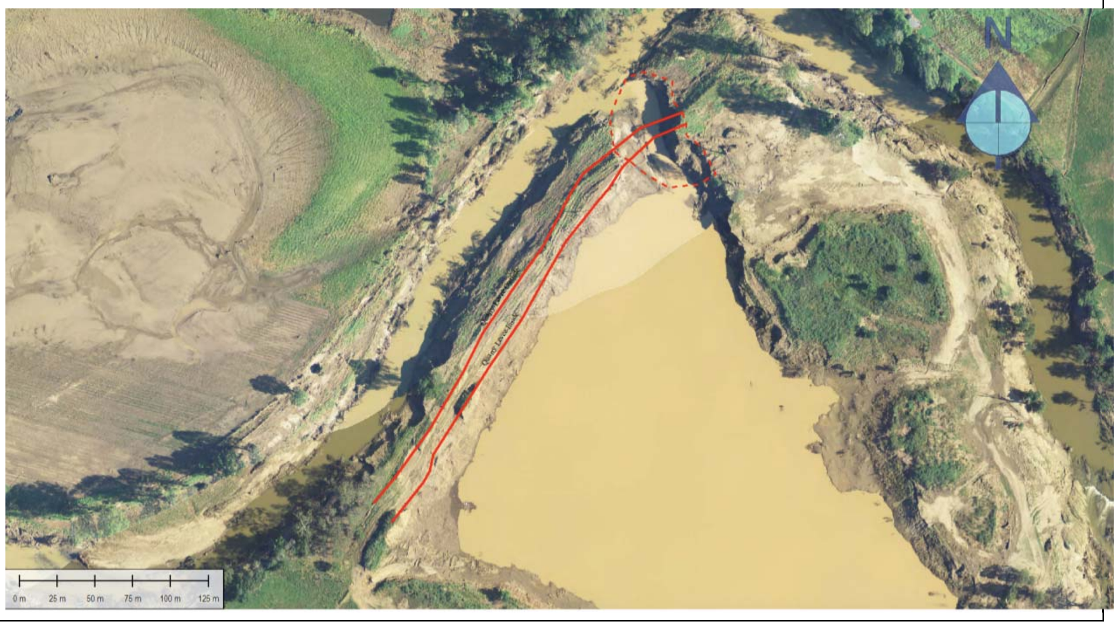


Figure 7.5 – Pre and Post Flood Erosion of Western Levee of Quarry

8 Preparation of the GFCOI Flood Model

8.1 Overview

151. I have used the LVRC models developed by Jacobs as a starting point for my investigations. As noted above, the model data provided included:

- hydrology model (RAFTS) – providing estimates of catchment runoff flow hydrographs for the 10th January 2011 event, to a relatively high spatial resolution, over the entire Lockyer Valley; and
- hydraulic model (TUFLOW) – providing estimates of floodwater surface levels and velocities (2D) for the 10th January 2011 event, to a 10m gridded resolution (Helidon-Grantham area), over the entire Lockyer Valley.

152. Jacobs (2014, Section 6) have reported in their flood study report that they calibrated both of these models to available data:

- RAFTS to recorded discharge at available DNRM stream gauging stations throughout the catchment, using estimates of rainfall derived from BOM rainfall recording stations and Rainfall Radar imagery; and
- TUFLOW to recorded peak flood heights (computer data file provided by LVRC) throughout the modelled area, using estimates of stream flow from RAFTS model outputs.

8.2 Suitability of the LVRC Models

153. Following my review of the LVRC TUFLOW model, I formed the opinion that it was not suited to the needs of the GFCOI. The key issue with the model was that it was designed for general flood hazard and development assessment purposes. Although it was calibrated to the 10th January 2011 flood, it did not make direct use of actual records of creek flows (e.g. at the Helidon Gauging Station). Instead, the model used calculated estimates of storm event flows derived from rainfall.

154. Another secondary issue was the pragmatic need for me to reduce the time it took to undertake a model simulation. I found that the LVRC model runtime was very long (close to 14 hours). I considered that I would need to reduce the runtime sufficiently in order to allow me to undertake the large number of simulations that I expected I would require to fully investigate the flooding in Grantham.

8.3 Adaption of the LVRC Models

155. I adopted a standard approach to adapt the LVRC models to meet my needs, which was to create a sub-model directly from the LVRC models' data. Metaphorically what I did was like using a pastry cutter to cut a shape out of a larger piece of pastry – but numerically. In doing so, I also needed to define conditions at the upstream and downstream ends of my sub-model to represent how the model interacts to the "outside world" at these locations. These are called "boundary conditions".

156. My new sub-model (GFCOI model) extends from the Helidon Gauging Station at the upstream to a short distance downstream from the Gatton-Helidon road bridge crossing of Lockyer Creek (approximately 5 km downstream from Grantham).

157. I have shown the extent of the modelled area in the GFCOI model in Figure 8.1. The area extends from the Helidon Gauging Station in the northwest, to near the Gatton-Helidon road bridge crossing of Lockyer Creek in the southeast, a distance of some 15km. The upstream boundary condition was the flow hydrograph from the Helidon Gauging Station, and the downstream boundary condition was a rating curve (i.e. relationship between water level and flow rate) for the creek and floodplain.

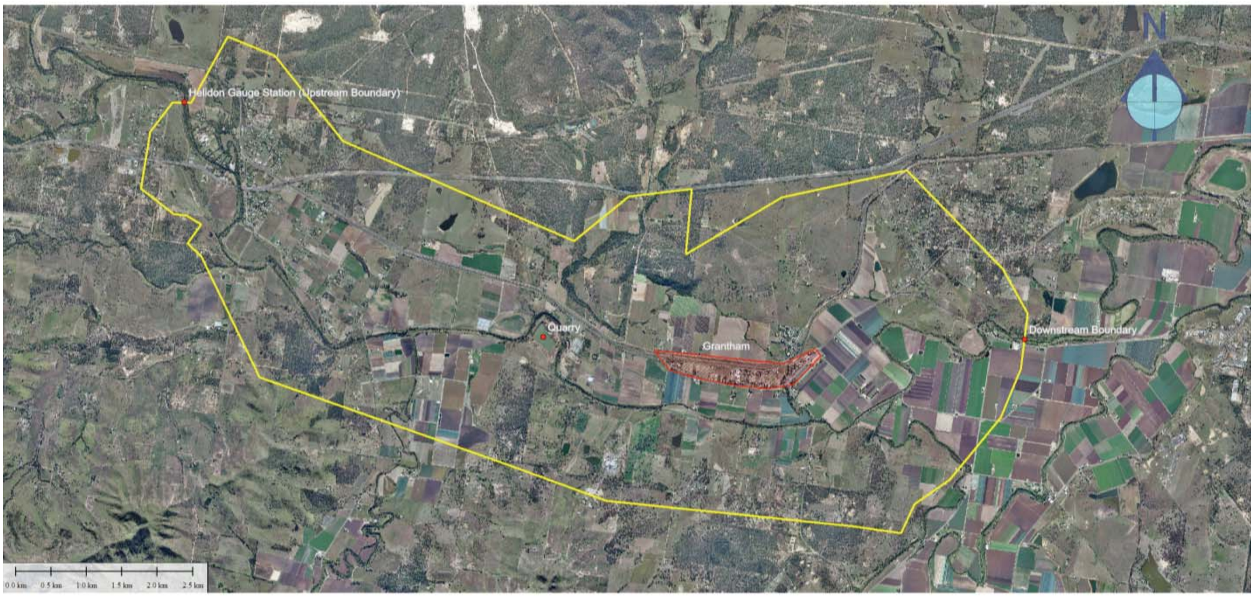


Figure 8.1 – Extent of GFCOI Hydraulic Modelling Area

158. My formulation of these boundary conditions is discussed in the following sub-sections.

159. Another change was to replace the inflow hydrographs in the model for Flagstone and Ma Ma Creeks, that had been estimated using the RAFTS hydrological model, with alternative hydrographs derived directly from Gauging Station records at those locations (refer to Appendix B.4 for details). This was done as I considered that it was better to use estimated inflows that were as closely aligned to actual records as possible.

160. In the following sub-sections I have provided more details of those items that I consider are of importance to the function of the GFCOI model:

- summary of the changes that I have made to the LVRC models in constructing the GFCOI model;
- the flow hydrograph at the Helidon Gauging Station (upstream boundary condition);
- the flow hydrographs at the Flagstone Creek and Ma Ma Creek Gauging Stations;
- the stage-discharge rating curve for the downstream boundary condition;
- representation of the Grantham Quarry at the time of the flood (including erosion), and before any development of its site; and
- model testing, including sensitivity.

8.4 Changes to the LVRC Model

161. In the previous section, I discussed my reasons for modifying the LVRC models to suit my purposes. I refer to this extract of the LVRC models as the GFCOI model. The changes that I have made to the LVRC models are summarized as follows:

- extract all relevant Geographic Information System (GIS) data within the extents for the GFCOI model (Figure 8.1) from the LVRC models, including:
  - topography (10m gridded resolution);
  - land classification (hydraulic roughness);
  - direct rainfall (10th January 2011 event);
  - local inflow hydrographs (e.g. Sandy Creek, 10th January 2011 flood event);
  - hydraulic structures (e.g. bridges, culverts);
  - features (e.g. roads, gullies, levees); and
  - special features (Grantham Quarry levee system).
- added new boundary condition locations:
  - a flow hydrograph at the upstream end (Helidon Gauging Station); and
  - an elevation-discharge rating table at the downstream boundary.
- replaced flow hydrographs for:
  - Lockyer Creek at the upstream boundary (Helidon Gauging Station)
  - Flagstone Creek local inflow; and
  - Ma Ma Creek local inflow.
- additional simulation output reporting scopes and locations.

162. I tested the performance of the GFCOI model (Section 12) and confirmed its satisfactory operation.

8.5 Helidon Gauging Station 2011 Flow Hydrograph

Overview

163. The Department of Natural Resources and Mines (DNRM) operates the Helidon stream gauging station. The station (143203C Lockyer Creek at Helidon 3) has been in operation since 1987 and is reliably rated up to a flow rate of 120m^3/s.

164. DNRM has provided a record of the stage height hydrograph (which records the water level and time) from this station for the 10th January 2011 flood event. It is my understanding that the station failed to record the peak of the flood because of instrument failure and that DNRM has replaced the missing data with an estimate of the peak height derived from post-flood survey. I am satisfied that DNRM has provided the best available estimate of this stage height information.

165. DNRM has also converted recorded stage heights into associated total flow rates within Lockyer Creek at the gauging station. It has done this using a rating table (or curve) that it has established. A rating curve is used to convert recorded heights to estimated flow rates. DNRM hydrographers have developed this rating curve for the Helidon Station based on the cross-section at the gauge and measured flow velocities across the cross-section during flow events (referred to as gaugings).

166. My investigations have led me to conclude that the DNRM rating table for Helidon is inaccurate. Its application to the 10th January 2011 flood results in an underestimate of the flood discharge.

167. I have also reviewed the reports prepared by Jacobs that accompanied the LVRC models. From these reports it appears to me that Jacobs had used an alternative rating curve for their estimation of the flood discharge hydrograph at Helidon. Appendix A of their report (Vol. 2, p A-1, 2014) shows a "Hydrologic Model Calibration Plot" for the Helidon Gauging Station. This plot indicates a peak discharge of approximately 4,200m^3/s. This conflicts with DNRM's peak discharge of 3,640m^3/s. No explanation appears to have been provided by Jacobs in their documentation for this work.

168. However, after examining the outputs from the LVRC models I found that the simulated stage height and discharge hydrographs at the location of the Helidon Gauging Station were inconsistent from that of Jacobs' hydrologic modelling, as I have summarized in Table 8.1 below.

Table 8.1 – Comparison between Jacobs (2014) and DNRM Hydrology, Helidon Station

Item	DNRM	Jacobs	
		Hydrological Model	Hydraulic Model
Peak Flow	3,640 m^3/s	~4,200 m^3/s	3,590 m^3/s
Peak Height	13.88 mGL	-	14.42 mGL
Time	3:10 pm	~3:00 pm	3:42 pm

169. I have also included DNRMs hydrograph data for stage height and time of peak in Table 8.1. Review of this information shows considerable inconsistency between the various data sets. I have considered this and concluded that a possible reason could be on account of an unresolved issue with the Helidon rating curve.

170. I also note that the DNRM rating curve is marked as "poor" by DNRM for any flow above the highest rated value of 120m^3/s, or above approximately 3% of the 10th January 2011 peak flow rate. The classification of poor means that the rating has not been confirmed through accurate field measurement.

171. The method that I have applied to resolve this issue is detailed in the following sub-sections, and is summarised as follows:

- review of the DNRM method of estimating the flow hydrograph;
- review of the LVRC method of estimating the flow hydrograph;
- derivation of a revised rating curve for the Helidon Station; and
- application of the revised rating curve for the estimation of a revised flow hydrograph.

<u>DNRM Estimate</u>

172. The Helidon station recorded a stage hydrograph of the 10th January 2011 flood. I obtained a copy of this data directly from the DNRM water monitoring data portal (web site) and have plotted it in Figure 8.2 below.

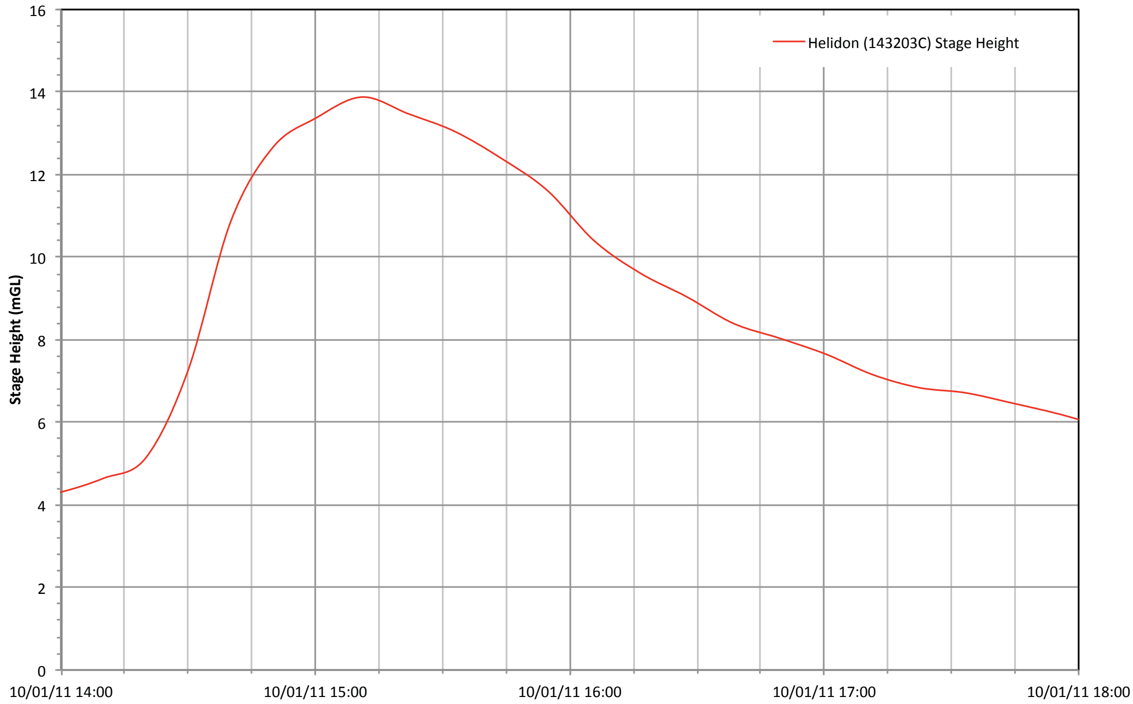


Figure 8.2 – Helidon (143203C) Stage Hydrograph 10 January 2011

173. As previously discussed, the rate of rise of the stage hydrograph at Helidon was very rapid. As seen in Figure 8.2 it had an average rate of about 1m in 8 minutes with a maximum rate of about 1m in 4 minutes.

174. DNRM has provided a rating curve for the station, which I have reproduced in Figure 8.3. Please note that the axes are plotted logarithmically. This is often done to make the plot seem more linear when plotted. However, a logarithmic scale is more difficult to read than a linear scale. I have re-plotted the same information in Figure 8.4, but on a linear scale.

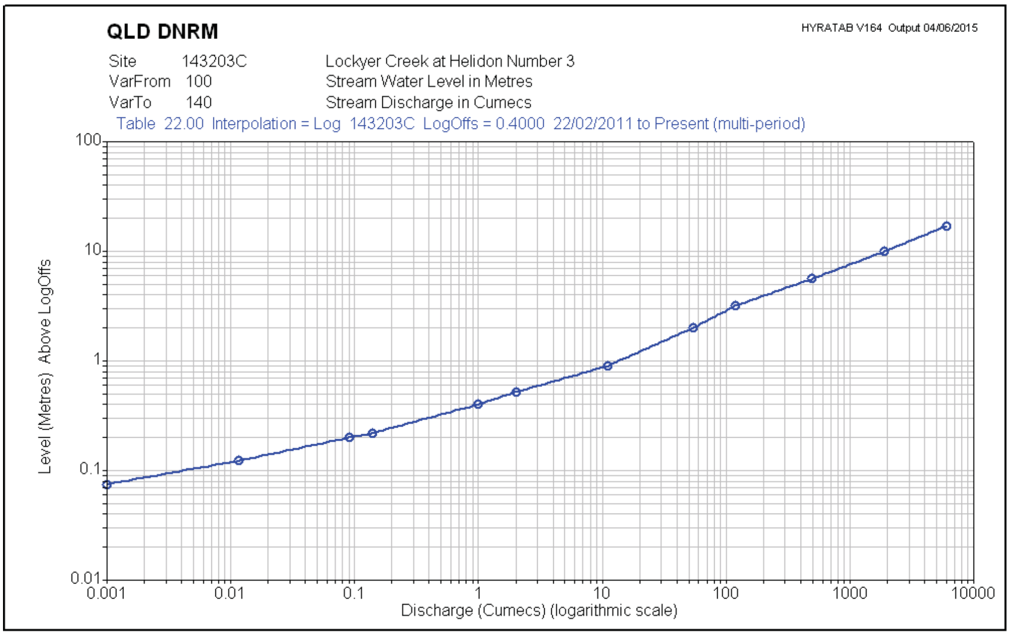


Figure 8.3 – Helidon (143203C) DNRM Rating Curve

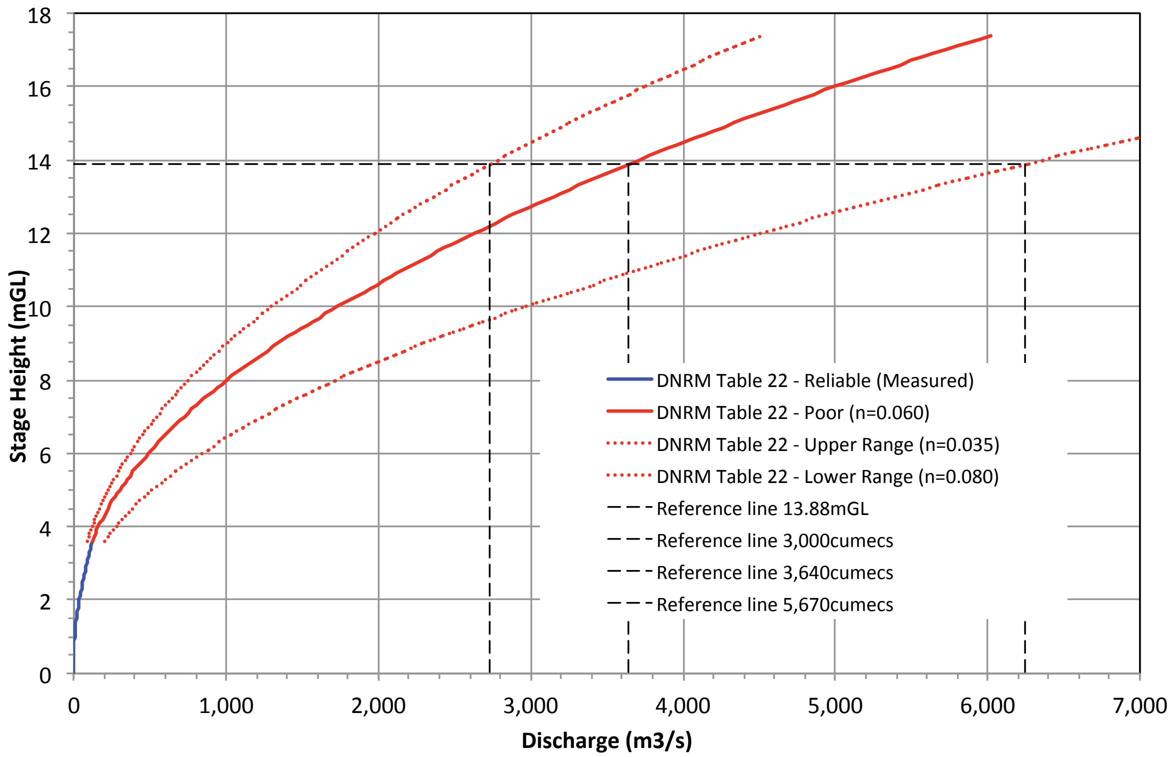


Figure 8.4 – Helidon (143203C) DNRM Rating Curve – Re-plotted Linear Scale

175. I have shown additional information in Figure 8.4 to better explain the substance of the DNRM rating curve.

176. First, I have indicated in Figure 8.4 that part of the rating curve that DNRM has classified as providing a reliable estimate of flow rate (blue line, between about 0 and 3.5mGL). This part of the curve corresponds to the range of actual field measurements of flow rate. It extends only up to a flow rate of 120m^3/s. This is not much when compared to a peak stage height of 13.88mGL that equates to a peak flow rate from the DNRM curve of 3,640m^3/s.

177. Second, I have shown the part of the rating curve above 3.5mGL as a solid red line. This part of the rating curve has been calculated by DNRM and, as indicated above, is a calculation that DNRM considers as being of poor reliability.

178. Third, I have shown two red dotted lines that represent my considered assessment of the likely range of uncertainty of the rating curve. My assessment has been rather simplistic and based around my appraisal as to how much I would reasonably expect the actual equivalent hydraulic roughness of the surface of the waterway might differ from a roughness estimated from visual inspection of photographs and experience (please refer to Appendix B.2 for more details).

179. On the basis of my assessment of the uncertainty associated with DNRM's rating curve it will be seen from Figure 8.4 that at the peak of the flood the actual peak of the flood flowrate was somewhere in the range of about 2,700m^3/s to about 6,200m^3/s (as shown by the black dashed reference lines). The DNRM gauging station value of peak flowrate sits in the middle of this range at 3,640m^3/s.

LVRC Models Estimate

180. Given the uncertainty above, I used the standard DNRM rating curve to convert the recorded stage height hydrograph at Helidon (Figure 8.1) into a flow hydrograph. The outcome is plotted in Figure 8.5 (solid red line).

181. For comparison, I have also plotted in Figure 8.5 the flow hydrograph at Helidon for the 10th January 2011 flood event from the LVRC hydraulic model simulation results (blue dashed line).

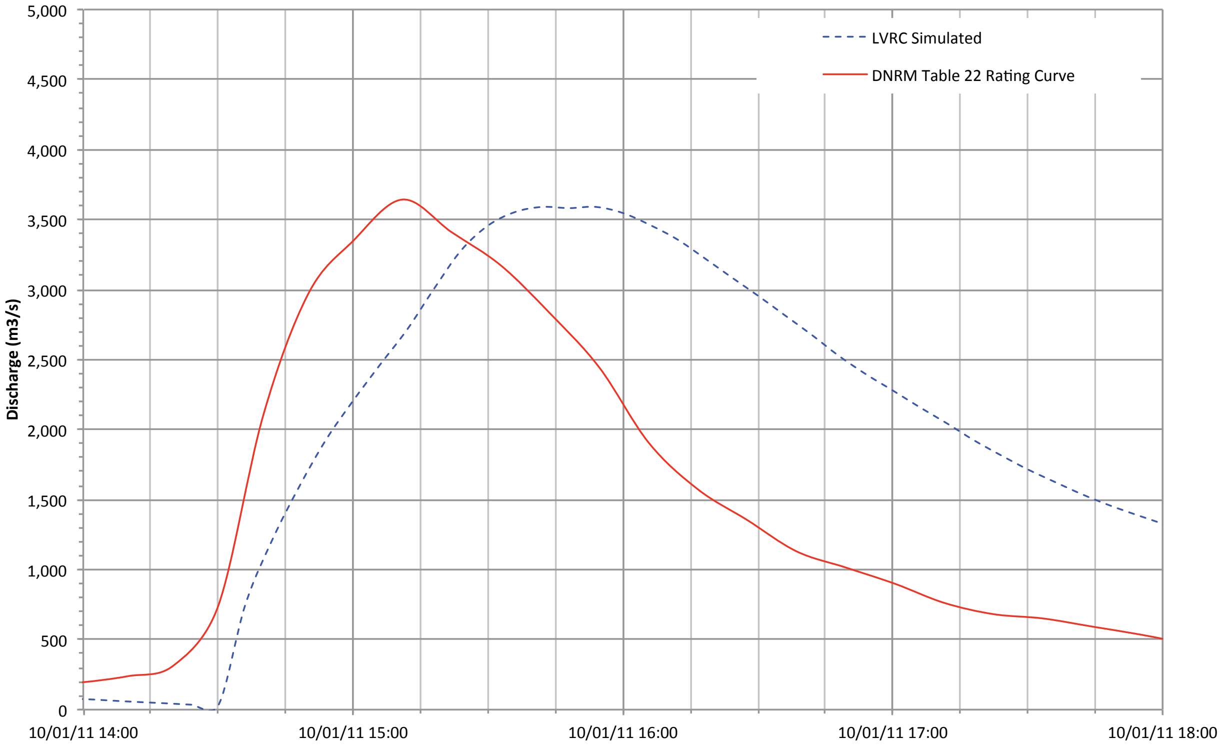


Figure 8.5 – Helidon (143203C) Flood Flow Hydrograph Estimates

182. Figure 8.5 shows the LVRC model simulated peak flood flow rate as very close to the DNRM estimate of 3,640m^3/s. To me this indicates that Jacobs calibrated their modelling to achieve this outcome and is consistent with my expectations.

183. However, Figure 8.5 indicates that there is a significant lag in time of a little more than 30 minutes between the LVRC model peak and that from the DNRM Gauging Station records for Helidon. I also notice that the volume of the LVRC hydrograph (represented by the area under the curve) appears to be significantly more than that under the DNRM hydrograph; about 30% more to the peak of the hydrograph. A further problem with the LVRC model hydrograph is that the slopes of the rising and falling limbs are significantly gentler than indicated by the DNRM hydrograph.

184. This mismatch indicates an inconsistency between the LVRC calculation and DNRM records.

Revision of Rating Curve for Helidon Gauging Station

185. My opinion is that there is a problem with both the LVRC model and the DNRM rating curve. It appears to me that in undertaking their calibration work Jacobs have probably calibrated their hydrology (RAFTS) model to their own version of a rating curve for Helidon, but failed to achieve good calibration of their hydraulic (TUFLOW) model, as indicated in Table 8.1.

186. If my findings are correct then fixing the problem would require that I undertake a complete and detailed review of all work undertaken by Jacobs in the preparation of the LVRC model, including recalibration of both their hydrological and hydraulic models. This is a considerable amount of work and would take me many months to complete.

187. As an alternative, I have applied a simple technique based around flow hydraulics (Manning's equation) to make approximate adjustment to correct the DNRM rating curve (please refer to Appendix B.2 for the technical details).

188. My revised rating curve for Helidon is presented in Figure 8.6. The original DNRM curve is also plotted in the figure for reference.

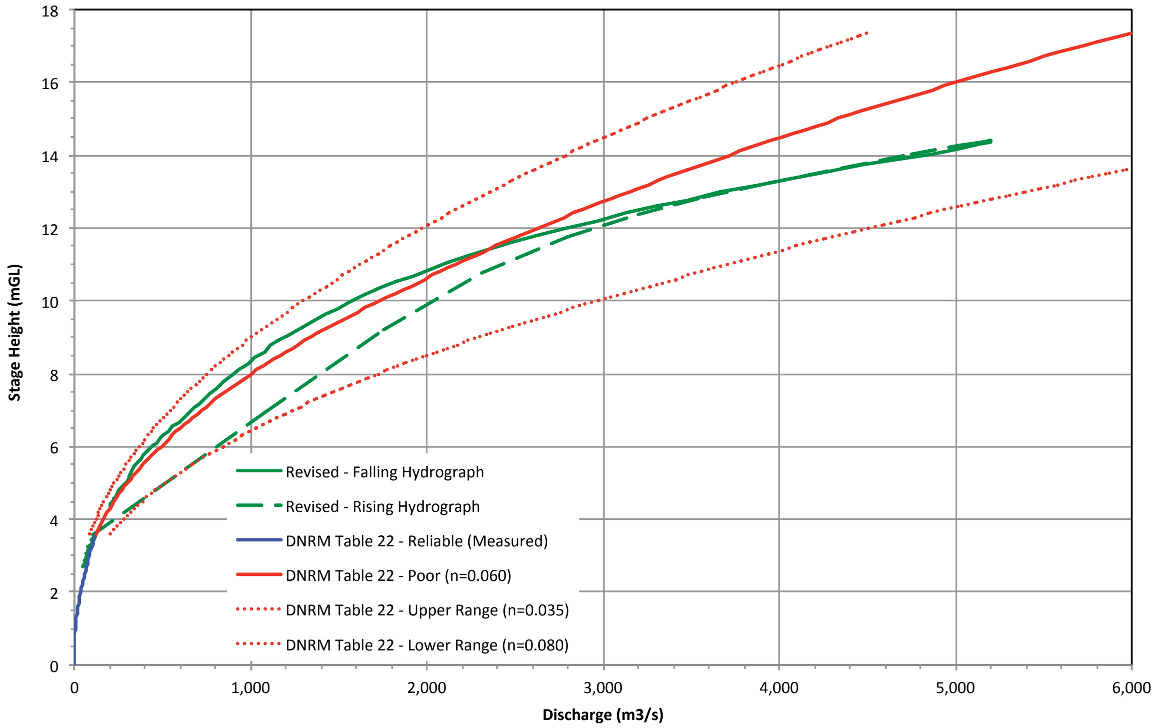


Figure 8.6 – Helidon (143203C) Revised Rating Curve

189. The first thing to note when reviewing the rating curves shown in Figure 8.6 is that I have produced two revised curves (plotted in green). The lower line (dashed green) corresponds to the rating relationship when the flood is rising, and the upper line (solid green) corresponds to when the flood is falling. This difference is known as hysteresis and is an established characteristic of a typical flow hydrograph. It comes about because the slope of the water surface is usually steeper when flood flows are increasing and vice versa when falling. I obtained data to plot this hysteresis characteristic at the Helidon Gauge from the LVRC hydraulic model, as discussed in Appendix B.2.

190. Figure 8.6 also shows that the new revised rating curve sits within the bounds of my previous (and independent) assessment of my expected range of uncertainty of the original DNRM rating curve, as discussed in Appendix B.2.

Revised Flow Hydrograph

191. Applying this revised rating curve to Helidon Gauging Station records of stage height during the 10th January 2011 flood then provided me with a corrected estimate of the flood flow hydrograph at the Helidon Gauging Station. This hydrograph is plotted in Figure 8.7 along with those from the original DNRM rating and the LVRC flood model.

192. Review of the information plotted in Figure 8.7 shows that the revised flow hydrograph at the Helidon Station has a peak rate of 4,600m^3/s, or approximately 25% greater than that from the original DNRM estimate. The flow volume of the revised hydrograph (to peak) is now the same as that associated with the LVRC simulated hydrograph. As discussed in Appendix B.2, this is an important match as this volume, by and large, is in effect the volume of the rainfall that had previously runoff from the catchment, and this rainfall was determined by Jacobs based on measurement.

193. The other very important characteristics of the revised hydrograph shown in Figure 8.7 are: the timing of the peak now matches the DNRM time of recorded water levels; and the shape of the hydrograph now seems to better match the original DNRM estimated hydrograph (shown as the red line in Figure 8.7).

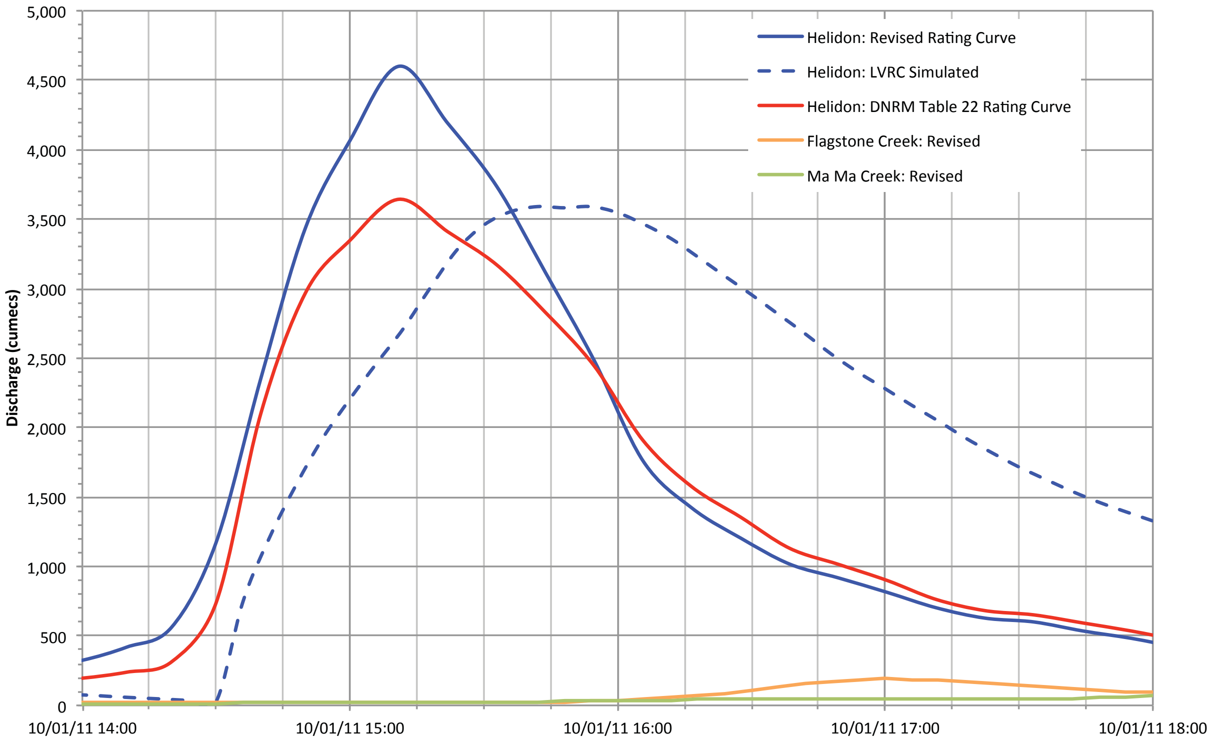


Figure 8.7 –Revised Flood Flow Hydrographs at Helidon, Flagstone and Ma Ma Creeks

8.6 Flagstone and Ma Ma Creek Flow Hydrographs

194. My review of the flow hydrographs used in the LVRC models for local inflows from Flagstone Creek and Ma Ma Creek revealed inconsistencies with gauging station records.

195. I have replaced the hydrographs provided in the LVRC TUFLOW model with alternatives that I have derived using DNRM's gauging station based flow hydrographs. I undertook my

assessment using a suitably adapted version of Jacobs' hydrological (RAFTS) model. Please refer to Appendix B.4 for details of the manner in which the model was adapted.

196. I have presented the resulting flow hydrographs in Figures 8.8 and 8.9 for Flagstone Creek and Ma Ma Creek respectively. The plots include the Jacobs' and DNRM's hydrographs for reference.

197. I have also plotted the revised hydrographs in Figure 8.7 for comparison with those of Helidon. I note that the peak of the hydrographs take approximately 1 hour to travel from Helidon to Grantham and this needs to be taken into account when comparing the timing of the flow hydrograph in Lockyer Creek at Helidon, as against that in Flagstone Creek and Ma Ma Creek.

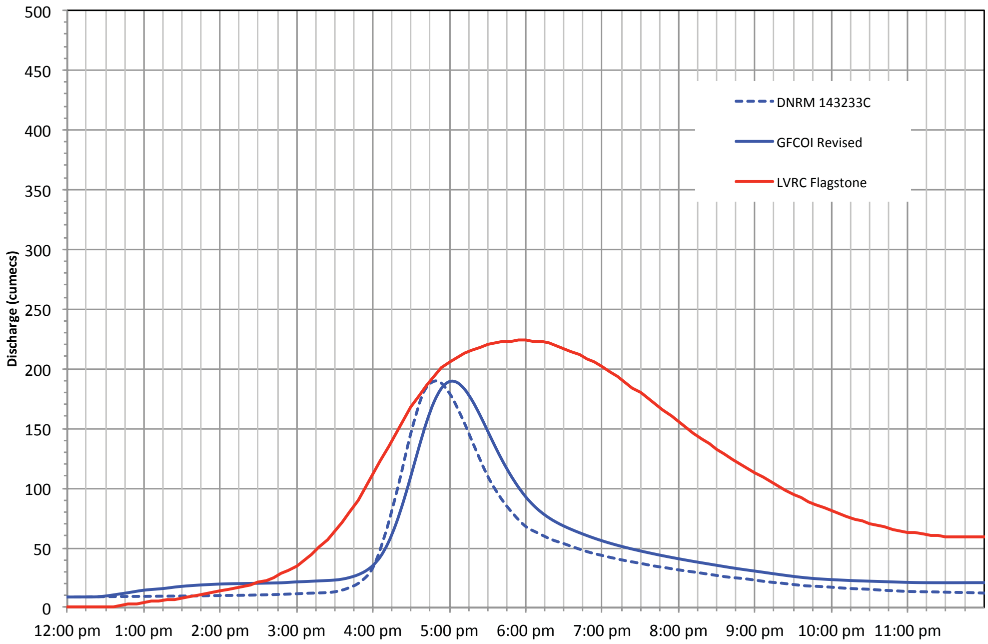


Figure 8.8 – Flagstone Creek Revised Flood Flow Hydrograph

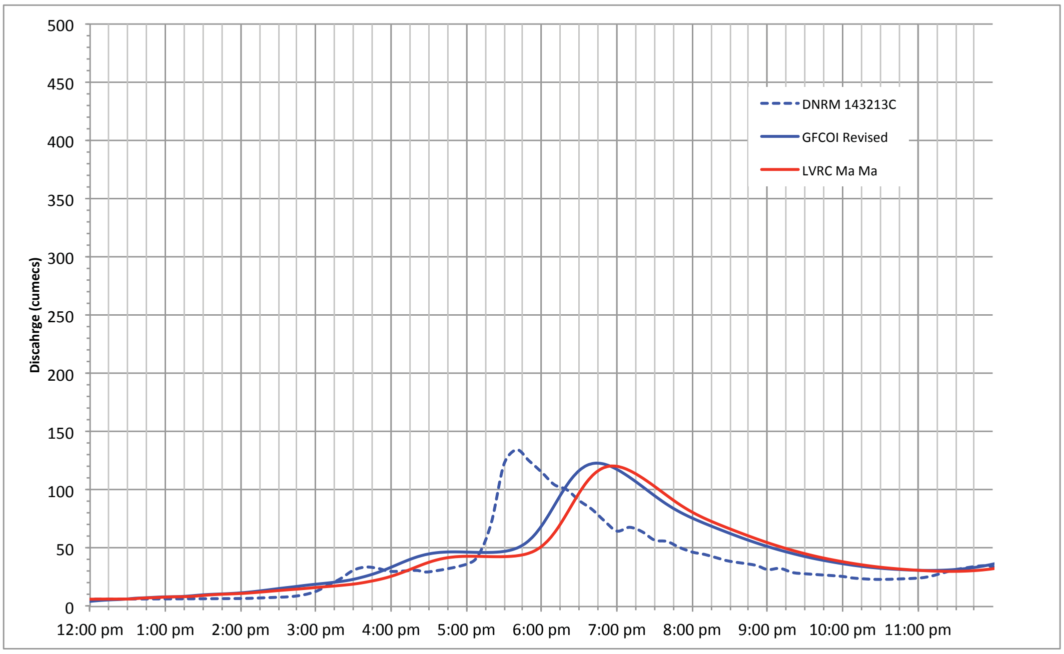


Figure 8.9 – Ma Ma Creek Revised Flood Flow Hydrograph

8.7 Downstream Stage-Discharge Relationship

198. I derived the required downstream model boundary condition of a stage-discharge relationship by sampling simulation results, at the location of the downstream boundary, from the LVRC model. This relationship is plotted in Figure 8.10 below. Please refer to Appendix B.5 for a detailed appraisal of the effect of downstream boundary condition assumptions on hydraulic model performance.

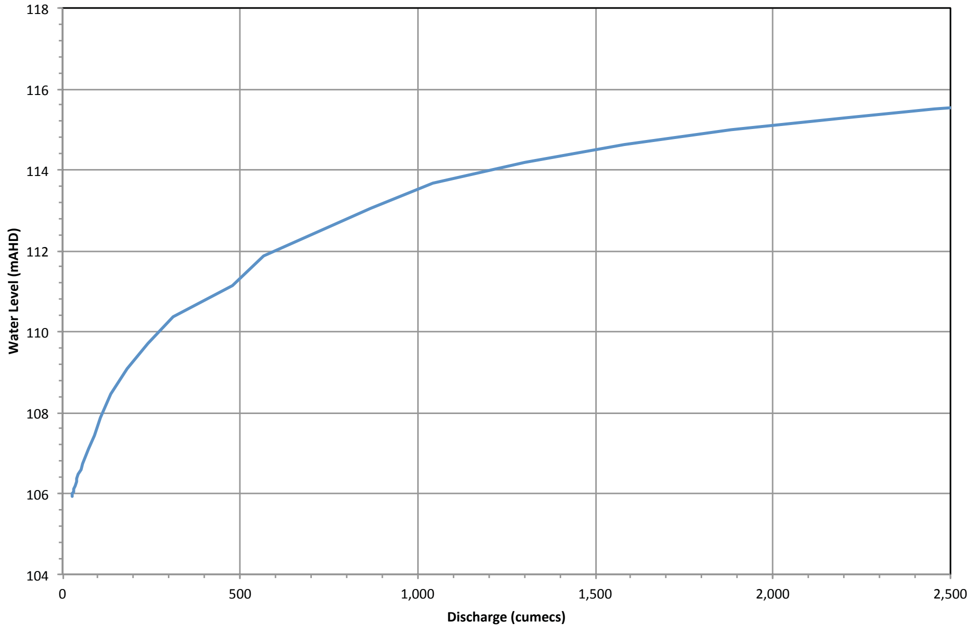


Figure 8.10 – Downstream Boundary Stage-Discharge Relationship

8.8 Modelling of the Grantham Quarry Breaches

General

199. I have provided a schematic cross-section through the Western Levee with feature labels for reference, Figure 8.11 (Starr, 2015).

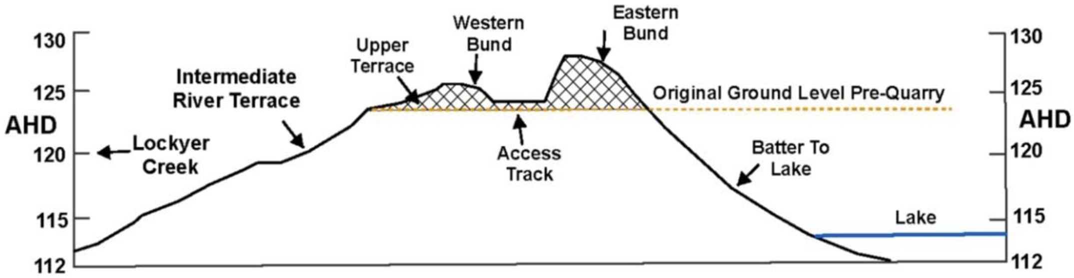


Figure 8.11 – Schematic Cross-section Through Western Quarry Levee (Starr, 2015)

200. I have also prepared an oblique image showing the Quarry as of August 2010 with annotations and labels of features, Figure 8.12.

Figure 8.12 – Oblique Showing Pre-flood Quarry as at August 2010

201. My oblique image (Figure 8.12) is for conditions prior to the occurrence of the 10th January 2011 flood and shows the pit surrounded by a system of bunds, spoil and stockpiles. Although I have not been able to identify any clear record of the line and level of these levees prior to the 2011 Flood Event, I have had the benefit of the opinion of Mr Starr (Geotechnical Expert). That opinion confirms my understanding that survey data captured by LIDAR aerial survey in August 2010 provides a reasonable representation of the quarry as at 10th January 2011.

202. The line of Lockyer Creek is also seen in the image. In my opinion, the system of bunds / stockpiles and spoil, in conjunction with the natural creek bank, serves to prevent smaller floods from entering the Quarry.

203. As discussed in Section 12.6, I consider that the bunds along the western side of the pit (Western Levee) that existed prior to 10th January 2011 would have restricted the flow of the flood in the Lockyer Creek waterway on the 10th January 2011.

204. Over the course of the flood, the bunds making up the Western Levee largely disappeared and a deep breach (down to creek bed level) appeared in the bank that separated the pit from the creek towards the northern end of the Western Levee. The locality and extent of this damage is indicated in another oblique view of the pit after the flood in January 2011, Figure 8.13. I have also described this breach as the Main Breach, as indicated in Figure 8.13.

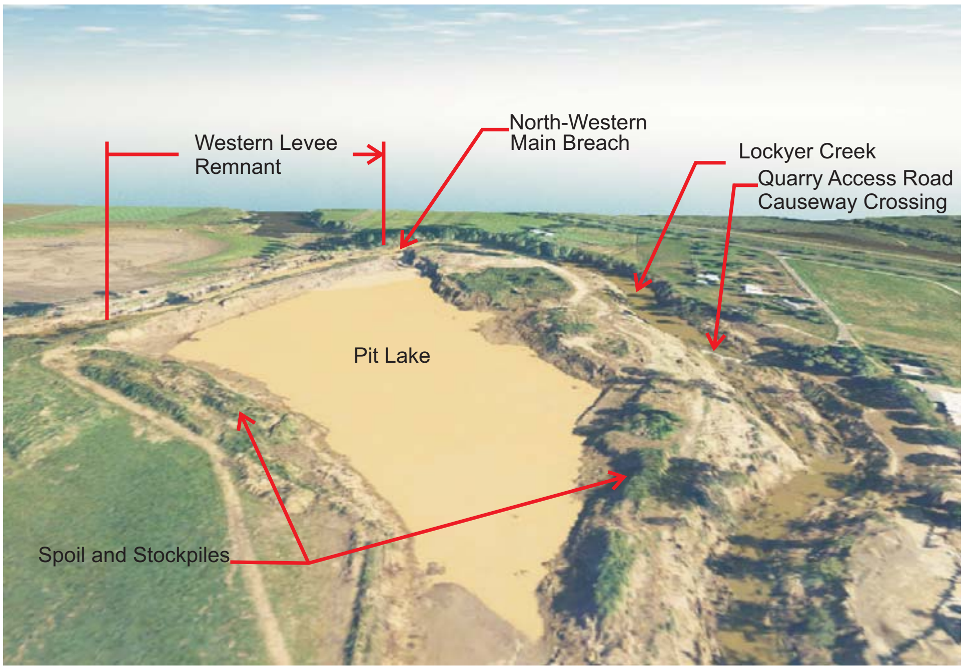


Figure 8.13 – Oblique Showing Post-flood Quarry as at January 2011

205. During my inspection of the quarry, I noted what appeared to be a relatively steep cliff face along the inside edge of the western side of the quarry pit. The nature of this face indicated to me the likelihood that a form of landslip failure had occurred at some stage during the course of the flood event. My inspection of the pre and post-flood aerial photographs indicated that the alignment of this slip face appeared to run close to the alignment of the original Eastern Bund in parts.

206. Figure 8.14 shows a photograph provided by Mr McIntosh (photograph DSCF7655.jpg) and taken shortly after the flood event on 12 February 2011. Looking at the image I see that it was taken from a location that overlooked the pit (eastern) side of the Western Levee looking north. Notable items in the image include:

- image centre, left: a centrally located parapet type remnant of the Eastern Bund, grassed on the left-hand side (west) to crest, bare earth steep and clean face to the right-hand side (east) and facing side (south);
- image front, left: what appears to have been a remnant of the original Eastern Bund now completely slumped into the pit, with what still appears to be the remains of a steep and clean face on its facing (south) side;
- image front to back, left: relatively smooth plain surface under the full extent of those bunds that had been washed away;
- image front to back, left: a relatively uniform, clean and sharp edge running the entire eastern length of the bank; and
- image front to back, left: a significant beach of what appears to be similar material to elsewhere in the image.

Figure 8.14 – Inside Western Levee Looking North as at 12 February 2011 (McIntosh 2015)

207. On the basis of the information I acquired from my inspection of the site, the photograph from Mr McIntosh, the post flood aerial images from LVRC and my experience, I have concluded that three primary actions likely contributed to the "disappearance" of parts of the Western Levee during the flood:

- erosion of the bunds from flood waters traversing the levee from left to right (into the pit), as indicated by the clean and smooth areas under which the bunds originally stood, and the steep sharp facing sides on the remnant bunds;
- erosion of the Main Breach by floodwater flowing into the pit; and
- slip failure along portions of the eastern side of the Western Levee.

208. Whereas the process of erosion happens progressively over a period of time, albeit sometimes relatively quickly, it is not unusual for slip failures to occur relatively rapidly (i.e. in a matter of minutes). Slip failures are also usually triggered by some external influence, for example the rapid draw down of water in creeks after a flood often results in bank slumping (a form of slip failure), or the undermining of the base of a steep batter can also initiate failure. I consider that both of these example mechanisms are equally applicable to the Grantham Quarry pit. However, I consider top down erosion as the most likely because:

- the occurrence of slip failure is usually on account of loss of material strength through saturation, or on account of foundation material giving away. I consider that these conditions are unlikely to have developed prior to the bunds being overtopped by the rapidly rising floodwater in Lockyer Creek; and

- slip failures more often occur on the recession of a flood when the water recedes from a saturated bank, and in my opinion, evidence of this type of post-flood failure is seen in the photograph.

209. As there are no eye-witness accounts of the breach occurrence, I am unable to ascertain with any certainty the exact detail of the timing and sequence of levee failure, be it through erosion, or slip failure, or both.

210. However, for the purposes of my investigations I consider that the actual mode of failure is not of critical importance. What I do consider important is:

- the initial ground surface profile prior to failure, and the final ground surface profile after failure;
- the time of triggering of commencement of failure, noting that different components of a levee system may initiate failure at different times; and
- the duration that it takes to fail.

211. I have examined the mechanism in the LVRC models for simulating failure of the Western Levee and have concluded that it provides the necessary detail and methods to reasonably represent the Western Levee system and its failure under flood conditions. I consider that it also provides a suitable basis with which to undertake a range of sensitivity analyses for the purposes of addressing uncertainty surrounding the timing and duration of the failure that occurred during the 10th January 2011 flood.

Modelling the Western Levee Failure

212. There are two bunds on the Western Levee of the pit, one either side of an access track. The alignment of these two bunds on the western (upstream) side of the Quarry is indicated in Figure 8.15 below (solid red lines).

213. In examining the LVRC models I have found that Jacobs have paid particular attention to the modelling of the subject Western Levee system failure. Key features of this mechanism include:

- before and after flood ground surface levels along the bunds of the Western Levee, and in the Main Breach area;
- the ability to define the duration of time over which the failure occurs; and
- water level height markers used to trigger commencement of levee failure in a progressive manner.

214. I consider that the method employed to model the failure mechanism is suitable for the situation and provides reasonable representation of breach conditions. Furthermore, the method is amenable to sensitivity simulation analysis, an important consideration for the purposes of my investigations.

Figure 8.15 – Alignment of Pre Flood Quarry Bunds and Location of Main Breach

215. During the course of the flood a significant breach scoured in the low point of the Western Levee (dashed red line) as indicated in Figure 8.15. The levee also suffered significant damage as well as creek banks and the inside of the quarry wall. An aerial photograph taken a short time after the 2011 Floods is shown in Figure 8.16 with same alignment lines marked.

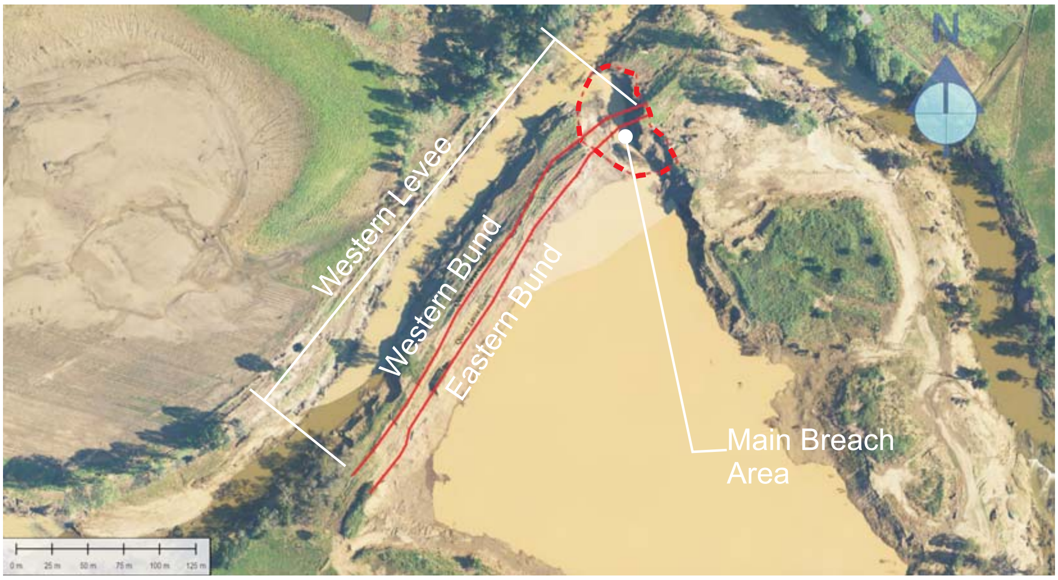


Figure 8.16 – Alignment of Post Flood Quarry Bunds and Location of Main Breach

216. I have extracted longitudinal profiles along the alignment of these bunds for both pre and post flood conditions from LIDAR data captured with the aerial photographs. These are plotted in Figure 8.17 and 8.18 for the Eastern and Western Bunds respectively.

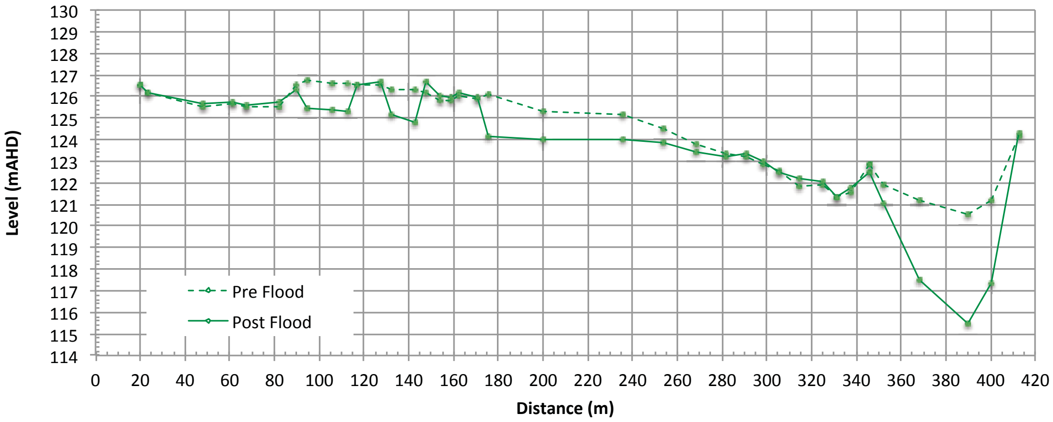


Figure 8.17 – Western Bund Profiles for Pre and Post Flood Conditions

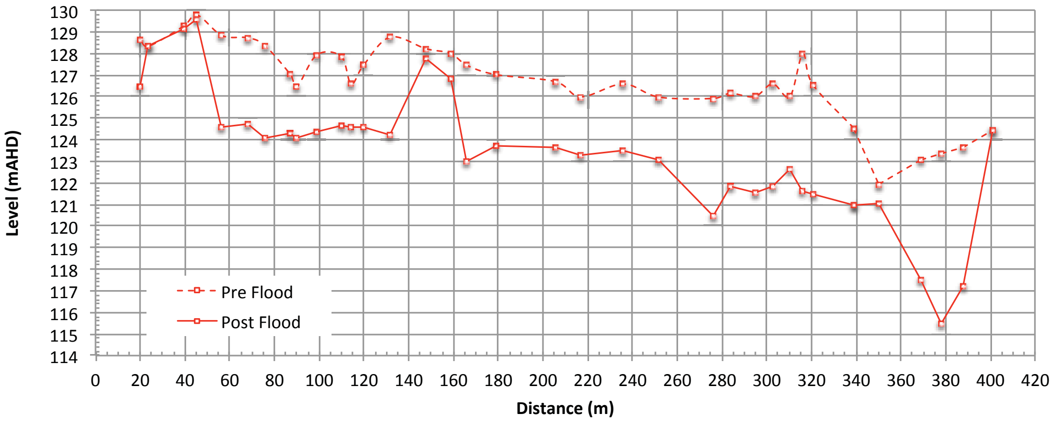


Figure 8.18 – Eastern Bund Profiles for Pre and Post Flood Conditions

217. In these figures, 0m on the horizontal scale is the upstream (southern) end of the alignment and the sharp dip in the post-flood profiles at 380m is associated with the Main Breach at the northern end of the levee. The green coloured lines are associated with the Western Bund alignment, and the red lines are associated with the Eastern Bund alignment. Dashed lines are pre flood and solid are post flood.

218. I notice from the plotted information that the Eastern Bund appears to have been generally higher than the Western Bund. A significant portion of the Western Levee seems to have been removed during the course of the 2011 Flood Event - up to around 3.4m from the bunds (excluding the Main Breach) and up to about 7.5m at the location of the Main Breach. It is also shown that post flood surface levels along the two bund alignments are close to the same.

219. As to the breaches of the Western Levee, I have observed six primary distinct areas of erosion that have combined during the course of the flood event to form three breaches into the pit on the Western Levee (totalling 330m in width), as listed in Table 8.2 below.

Table 8.2 – Western Levee Breach Locations

Location	Distance From (m)	Distance To (m)	Length of Breach (m)
Southern Breach	60	140	80
Central Breach	160	350	190
Main Breach	350	410	60
		Total	330

8.9 Quarry Pit Lake Levels

220. In my view, the level of water that was within the quarry pit immediately prior to its inundation by the 10th January 2011 flood event had a direct effect on the influence that the quarry exerted on the surrounding flood flows and levels.

221. There is only limited data providing direct measurement of the Quarry lake water level, but at different times to the flood:

- August 2010 LIDAR: 114.0mAHD (bed level in adjacent Lockyer Creek)
- January 2011 LIDAR: 115.4mAHD (water level in adjacent Lockyer Creek via breach)

222. The primary reason for the importance of lake level is the lower the level, the greater the available capacity to absorb (or store) flood inflows into the quarry in consequence of levee breach. The significance of available capacity is not only that this volume is effectively removed from the flood, but also that outflows from the pit area back into the downstream Lockyer Creek waterway will only commence once the pit has filled.

223. This means that while the pit is filling the flow hydrograph downstream of the quarry is delayed and reduced in magnitude.

224. I have undertaken an investigation into lake water levels by first deciding what mechanism would most likely control lake level, and then on the basis of this mechanism, made an assessment of an appropriate water level.

225. I note that the August 2010 surveyed level corresponds to the bed level of Lockyer Creek. Recognizing that the Grantham Quarry is for sand and gravel, a relatively permeable material, I consider it reasonable to expect that a strong hydraulic connectivity exists between the pit lake and the creek. This means that pit lake levels will generally follow water levels in the adjacent Lockyer Creek, with some dampened time lag.

226. I used historical flow rate data from the DNRM Helidon gauging station to estimate the average flow rates in Lockyer Creek over the days prior to 10th January 2011. I then converted these flow rates into estimates of water levels in Lockyer Creek adjacent to the western side of the Quarry. I did this by using a rating curve relationship for Lockyer Creek at the quarry that I derived from the original LVRC models. Outcomes are listed in Table 8.3 below.

Table 8.3 – Lockyer Creek Flow Rates and Levels at the Quarry Prior to 10th January 2011

Averaging Duration (days prior)	Creek Flow Rate (m^3/s)	Water Level (mAHD)
7	50	117
31	30	116

227. The information in Table 4.2 shows that the average flow rate over the preceding 31 days is less than that over the preceding 7 days, and this equates to a difference of 1m in estimated lake level. I note that the August 2010 and January 2011 LIDAR survey levels both corresponded to times of no flow in the creek, and I therefore expected them to be lower than the "with flow" levels listed in Table 8.3.

228. I have also obtained totals of daily rainfall from Helidon Post Office (Station 040096) over the periods prior to the 10th January 2011 flood as listed in Table 8.4 below.

Table 8.4 – Rainfall Totals Helidon Post Office (Stn. No. 040096)

Averaging Duration (days prior)	Period	Total Rainfall (mm)
7	3rd January 2011 to 9th January 2011	123
31	6th December 2010 to 9th January 2011	503

229. I have considered the possible contribution of rainfall to the quarry lake pit based on the Helidon PO records (Table 8.4) and, ignoring seepage and evaporation losses, selected a maximum pit level rise due to rainfall of 0.8m (assuming a total catchment for the pit lake of 16ha and a pit lake surface area of 10ha, and 100% rainfall runoff).

230. Based on the above I have consider a reasonable estimate of the quarry pit lake level immediately prior to the 10th January 2011 flood as:

- 117mAHD (highest in Table 8.3);
- plus 0.8m (highest rainfall contribution); and
- giving 117.8mAHD.

231. I have adopted a higher level of 120.0mAHD (an additional 2.2m) for the purposes of my investigations so as to further reduce the free flood storage capacity of the pit. In this way I have accounted for any other possible sources of volume contributions (such as, additional rainfall or further seepage inflows from Lockyer Creek).

232. I note that this level is also about 2m below the level at which I would expect outflows from the pit to occur and leaves a free storage capacity of approximately 200ML.

8.10 Carpendale Weir

233. There is a weir approximately 3m to 4m high located within the main channel of Lockyer Creek adjacent to Klucks Road. It is located immediately adjacent to the McIntosh residence on Klucks Road as indicated in Figure 8.19 by the red circle.

Figure 8.19 – Locality of Carpendale Weir

234. My detailed inspection of the modelling of the Lockyer Creek waterway upstream of this location found that the topography in the model matched that of the level water surface (the pond) behind the weir.

235. The original LVRC models based its topography directly on data from the August 2010 LIDAR aerial survey data. LIDAR data is not able to penetrate water. It picks up the water surface and not the ground surface beneath.

236. Hydraulic modelling requires good representation of the ground surface and adjustments therefore need to be applied to correct any issues associated with LIDAR reflections of water surfaces.

237. In the context of the 10^{th} January 2011 flood, the flow capacity of the Lockyer Creek channel, relative to that of the floodplains at the location of the weir is relatively small. The error in modelled main channel bed level upstream from the weir would only have some local effect on flood levels when flows were in-bank.

238. However, to compensate for this problem, I made an adjustment of the GFCOI model by lowering the modelled bed level to match the bed level of the creek immediately downstream from the weir and at the Kapernick's Bridge crossing (which seemed to be the upstream extent of the weir pool).

8.11 Sediment and Sand Deposits in Creek

239. From experience, I find that deposits of sands and sediments often accumulate within the waterways that are in close proximity to industrial operations that involve disturbance of the land.

240. I have considered the consequences that may arise if sediments and sands had been washing off the quarry industrial area and accumulating within the creek prior to the 10^{th} January 2011 flood event. Specifically I have considered if this would have affected the flooding of Grantham and surrounds.

241. I have undertaken a comparison between pre-flood (August 2010) and post-flood aerial photographs taken in January 2011 (LVRC 2014). That comparison depicted scour damage inflicted on the Lockyer Creek waterway in the vicinity of the quarry. This is consistent with my own observations during my site inspection that identified similar scour damage as depicted in the January 2011 aerial imagery.

242. In my view this clearly demonstrates the capacity that the 10^{th} January 2011 flood flows had to readily move sediment within the main channel of the waterway.

243. I consider that it is extremely unlikely that, had any sediment and sand accumulations been in the Lockyer Creek adjacent to the quarry prior to the flood, it would have had an effect on flooding.

244. Given this, I did not find it necessary to take any specific account of sand and sediment deposits in my simulation modelling.

245. I also make note that my proposition of there being sediment and sand on account of the nearby quarry is speculation only.

8.12 Obstruction to Flow by Debris and Buildings

246. Debris usually floats on the surface and can sometimes cause an apparent bulking of the surface with debris material that, to the observer, may be difficult to differentiate from the water. As a

consequence, to an observer, the addition of debris can confuse the correlation between observation and model simulation.

247. Further, to affect flood flows debris has to either:

- have sufficient concentration in the flow to introduce additional frictional and / or momentum losses; or
- collect sufficiently against objects protruding out of the flow in such a way as to restrict the flow through obstruction; such as against buildings, bridges and fences, but over sufficient area of flow so as to make a significant effect.

248. I have considered the presence of debris in the 10th January 2011 floods as described in eye-witness accounts, and shown in supporting photographs and videos. From this I observed:

- that the flood waters were carrying a reasonable load of debris; and
- there were debris accumulations on structures (including, but not limited to, Kapernick's Bridge) which had local effects on flood flows in the immediate vicinity of these accumulations.

249. In my view:

- the amount of the debris was relatively small in concentration when compared to the intensity of water flow so as to independently introduce additional friction or momentum losses;
- the debris accumulations were not sufficient to produce significant obstruction to flow; and
- accordingly the presence of debris observed in the 10th January 2011 floods was not sufficient to affect the net depth, intensity or timing of the flooding.

250. In any event, the intensity of water flow during the flooding in Grantham was significant. This means that even without debris loading I consider that much of the damage to structures I observed would have still occurred.

251. Given this, the 2D hydraulic model that I have used (GFCOI model) does not specifically model the presence of the debris.

252. However, I have also observed that the passing of the flood destroyed a number of houses and buildings. I have taken account of this occurrence within the GFCOI model by:

- including simulation of the blockage effect caused by the presence of flood affected buildings in the model;
- identifying those buildings that had been destroyed by comparison of pre and post flood aerial photographs (provided by LVRC);
- setting the model to progressively remove the blockage effect associated with those buildings subject to destruction using simulated flow depth as a basis to initiate removal.

8.13 Calibration Adjustment

253. Hydraulic roughness is a primary parameter that represents the roughness of the surface over which the water flows. Although recommendations are available (e.g. Engineers Australia, 2012), they are guidance only. Calibration of a hydraulic model is always required with hydraulic roughness being one of the primary parameters used to achieve calibration. In this regard, the

process of calibration usually involves the adjustment of hydraulic roughness until an acceptable match between simulated and recorded peak flood heights is achieved.

254. As a starting point, I reviewed the set of hydraulic roughness parameters derived by Jacobs (2014) from their calibration of the LVRC model to the 10th January 2011 flood. I formed the opinion from this review that the parameters they had selected were reasonable.

255. However, Jacobs' calibration of the LVRC model was on the basis of their estimate of the 10th January 2011 flow hydrograph which, as discussed in Section 8.3, differs from the revised hydrograph that I have developed for the purposes of my investigations. This meant that I needed to recalibrate the GFCOI model.

256. My recalibration work resulted in adjustment of the hydraulic roughness assigned to the "Farmland" land use classification from a Manning's n (hydraulic roughness) value of 0.05 to 0.06. I consider that this change in roughness is within an acceptable range for this type of land use. The net effect of this recalibration was to produce an overall average difference in peak height of 0.0m.

9 Chronology of the 2011 Flood Event in Grantham

9.1 Overview

257. At the time of the 10th January 2011 flood event, the presence of the levees, bunds, quarry spoil and stockpiles at the Grantham Quarry constricted out of bank flood flows down the Lockyer Creek. This constriction resulted in a range of change to flood characteristics, compared to what would have been the case had the quarry not been there. The most significant changes would have been:

- an increase in flood levels upstream of the quarry, most noticeably in close proximity of the quarry; and
- the diversion of additional flood flow rates around the quarry to both the north and south.

258. Flooding characteristics downstream of the quarry would have also been affected, including:

- changes in rates of flow and their distribution within the Lockyer Creek valley (i.e. between the main creek channel and overbank flow areas);
- changes in flood levels; and
- changes in the time of arrival of the inundating floodwater.

259. The magnitude of these changes to the downstream, like those for the upstream, varies with proximity to the quarry with diminishing effects as distance increases.

260. While the quarry levee banks remain intact, and floodwater is excluded from entering the quarry, the process that drives these changes is relatively straight forward. I summarize as follows:

- there is no change while creek flows remain in-bank;
- as flood flows break out onto the flood plains water begins to push up against the quarry levee on its upstream (western) side;
- the floodwaters against the levee cannot go through it and instead must go around to the north and south;
- this “redistribution” of flows pushes more water into the northern and southern flow paths than what would have otherwise gone along those paths had there been no quarry; water levels therefore increase in compensation to create a greater “head” (pressure) to force additional flow along these northern and southern flow paths;
- this increase in water levels also lifts the incoming floodwaters further upstream, but not all by the same amount; the increase in water level is greatest at the point of constriction (i.e. the Western Levee of the quarry) and then reduces upstream to a point when it becomes negligible (for the 10th January 2011 flood I estimate this to be around Kapernick’s Bridge, about 2km upstream of the quarry);
- as upstream flood levels are increased because of the quarry constriction, water that would have otherwise flowed downstream is temporarily stored upstream; this mechanism is called “flood detention” and results in a temporary reduction in the rate of flow past the constriction;
- hence, the net effect of the quarry levee, other than to keep floodwater out of the quarry pit, is to locally raise upstream flood levels and reduce the rate of flow passed to the downstream;

- immediately downstream of the Western Levee, but still adjacent to the quarry, flood flow rates would still be higher than what they would have been had the quarry not been there; flood levels and flow velocities would be higher;
- to the north of the quarry, an increase in flood levels might result in more overbank flood flows being diverted along the Western Overbank flow path towards Grantham than would otherwise have occurred had the quarry not been there, but only if the flood levels at the northern breakout point at the quarry hair pin bend are sufficiently high to force floodwater down this path; and
- reduced flow rates to the downstream of the quarry will reduce flood levels to what would have been the case had the quarry not been there.

261. However, the Western Levee did not remain intact (see Sections 6.3 and 7.3). Although there are not direct observations attesting to when the levees failed during the event, my opinion is that they most likely failed well before the peak.

262. The action of levee failure adds an additional level of flood hydraulics into play that also affects flood characteristics both upstream and downstream of the quarry. I also summarise these as follows:

- as the Western Levee of the quarry fails, floodwater held back behind it commences to flow into the pit;
- under these types of situations once failure of a levee starts (be it by overtopping scour, or landslip, or both) it typically develops over a relatively short period of time;
- although the rate of inflow is usually large, in the early stages the maximum rate is usually governed by the width of the breach in the levee through which it must flow and the depth of water over the bottom of the breach (and not the height over which the inflowing water must fall, at least not until later in the event);
- the pit must first fill before floodwater can commence discharging from the other (downstream) side; this filling completely removes the inflowing water from the flood, but only while filling to the point of overflow;
- in consequence, if there is no change to the rate of floodwater approaching the upstream of the quarry, then while the pit is filling all flood flow rates past the quarry would reduce, and so too would the resultant level of floodwaters downstream;
- however, if the approaching flood flow rates are continuing to increase at the time of levee failure (and pit inflow) then the effect of a reduction in flood flow rates past the pit can be masked;
- mind, while the pit is full, flood levels upstream from the quarry would still be higher than had the quarry not been there, and the opposite in the downstream;
- when the pit fills, flood flow rates downstream of the quarry increase, initially by an amount close to the rate of inflowing water into the upstream side of the pit (at the time of the pit becoming full) and then more slowly in concert with flood levels adjusting themselves to the presence of the new flow path through the pit;
- the largest change in flood levels after the pit fills will occur where the pit outflows into Lockyer Creek, that is, immediately downstream from the quarry; the rate of change in flood level at this location would also be relatively large in concert with the large change in flow rate

at this same location; upstream water levels in the immediate vicinity would also be lifted; to a bystander the rate of increase in flood levels in the local vicinity adjacent to and downstream of the quarry would seem very rapid after the pit fills compared to rates of increase prior to the filling of the pit; and

- the effect of changing flow rates (decreasing and then increasing), due to levee breaching and pit filling, would propagate downstream with the passage of the flood with its magnitude reducing over distance, an effect called “flood routing”.

263. A bystander downstream of the Grantham Quarry is thus likely to observe the following:

- the flood level rising within the Lockyer Creek channel and starting to spill over the banks;
- a reduction in the rate of rise as the quarry levee breaches and the pit is filling; and
- an increase in the rate of rise once the pit is full.

9.2 The Event

264. The flooding in Grantham on the afternoon of the 10^{th} January 2011 appears to have commenced about 3:15pm with Sandy Creek breaking its banks within Central Grantham adjacent to the Gatton-Helidon Road Bridge. This was approximately an hour after water level records from the Helidon Stream Gauging Station started to rapidly increase. At 3:15pm, the flood levels in Helidon had just reached their peak and, as indicated by eye-witness accounts, warning messages were already spreading by telephone and person-to-person. Yet, in Grantham there was little physical sign of the event to come.

265. From this time onwards floodwaters rapidly inundated Grantham in a manner that largely caught many people by surprise. This may well have been on account of not only the speed at which inundation occurred, but also the direction and intensity of flood flows which had not been previously experienced by the eye-witnesses.

266. From my investigations I have formed an opinion of the most likely chronology of the Grantham flood. I have quantified the timing and magnitude of the flood with the aid of the GFCOI hydraulic model that I constructed specifically for the purposes of my investigations. The GFCOI model is by no means perfect as it is merely a schematisation of reality. However, I have tested its performance and established the validity of its simulation outcomes with the accounts of eye-witnesses and measurements of peak flood levels from after the event. In this regard, I have also paid particular attention to the time of occurrence of events associated with the flood, most of which were associated with the very early stages of inundation. Details of my testing of the GFCOI model are presented in Section 12 of my report. Details of my corroboration of the GFCOI model with eye-witness accounts are presented in Section 13 of my report.

267. In the section that follows I present my assessment of the overall flooding sequence with the use of time-stamped simulation graphic panels depicting the extent of inundation and the intensity and direction of flows. The intensity of flow is indicated by a colour scale in units of m^2/s (velocity times depth) and arrows depict the magnitude of velocity and direction. A longer arrow means a larger velocity.

268. I note that the actual simulation modelling was undertaken at a very high time resolution with outputs captured to data file every minute. I have selected the time-steps used in Figures 9.1 to 9.3 below to provide reasonable coverage of significant flood characteristics while maintaining a consistent framework to assist with interpretation.

269. In conjunction with this report I have provided separately, movie files that present animation of complete simulation outputs for:

- the extent of inundation;
- flow intensity;
- velocity direction; and
- velocity magnitude.

These simulation sequences cover the period over 12pm to 8 pm on the 10th January 2011. The movie files have been produced with a time-step of 1 minute.

The file names and descriptions are listed in Table 9.1 below.

Table 9.1 – Simulation Movie Files

File Name	Contents
GFCOI_Most_Likely_Jan_2011.avi	Most Likely Case
GFCOI_No_Quarry_Jan_2011.avi	No Quarry Case
GFCOI_Worst_Case_Drop_Jan_2011.avi	Worst Case (greatest drop)
GFCOI_Worst_Case_Delay_Jan_2011.avi	Worst Case (greatest delay)
GFCOI_No_Rail_Jan_2011	No Grantham Rail Embankment

9.3 Summary Observations

270. As I have discussed above, the figures contain time-step *snap-shots* of simulation graphics for the most likely scenario at the selected areas and times listed in Table 9.2.

Table 9.2 – Index to Most Likely Chronology

Table	Location	From	To	Time-step (minutes)
Figure 9.1	Grantham Quarry and upstream	2:45pm	3:15pm	15
		3:20pm	3:45pm	5
		3:50pm	4:20pm	15
Figure 9.2	Central Grantham	3:25pm	3:55pm	15
		4:00pm	4:25pm	5
		4:30pm	5:00pm	15
Figure 9.3	Western Grantham	3:25pm	3:55pm	15
		4:00pm	4:25pm	5
		4:30pm	5:00pm	15

271. I have reviewed the information presented in Figures 9.1 to 9.3 and have also included my brief commentary that summarizes selected characteristics to assist the reader in their interpretation of the information. Enlarged copies of these figures are contained in Appendix C.

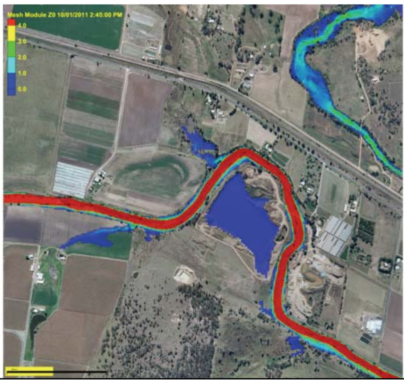
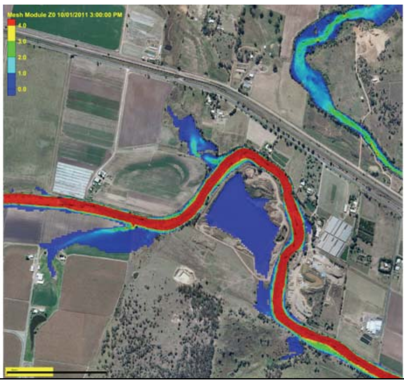
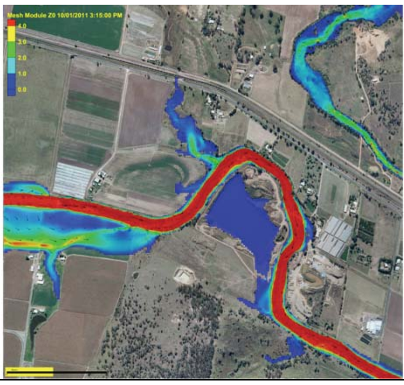
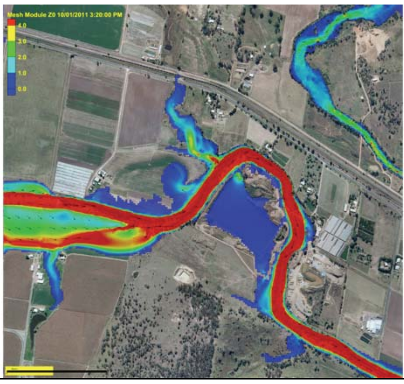
Image	Time and Commentary
	(a) 2:45pm • Flow within Lockyer Creek.
	(b) 3:00pm • Flow within Lockyer Creek at bank full at the upstream.
	(c) 3:15pm • Lockyer Creek breaking out of the waterway upstream from the quarry.
	(d) 3:20pm • Flow from Lockyer Creek spilling into the quarry (commenced in the minutes prior). Note inflow occurring in both the north-western and south-eastern quadrants of the quarry. • "Oval" area upstream of quarry receiving inflow.

Figure 9.1 a to d – Chronology of Inundation, Quarry and Upstream 2:45pm to 3:20pmTable

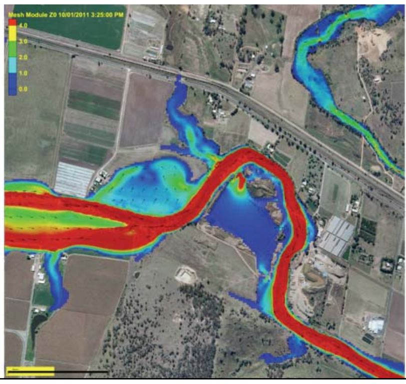
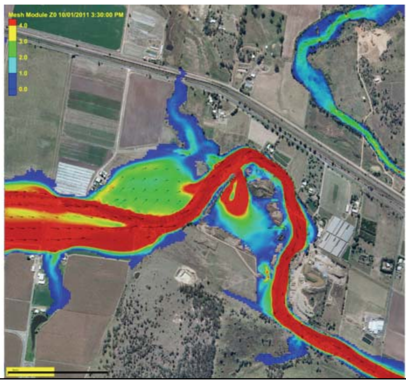
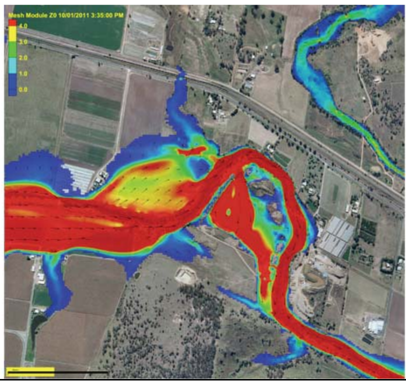
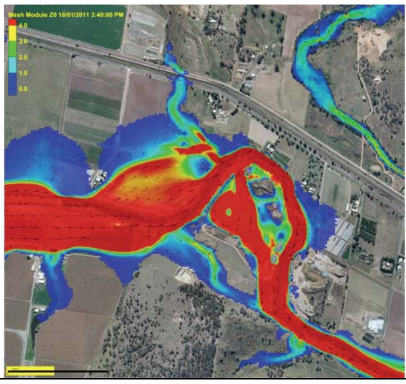
Image	Time and Commentary
	(e) 3:25pm • Lockyer Creek has broken out upstream from the quarry. • "Oval" area upstream of quarry receiving inflow.
	(f) 3:30pm • A marked increase in intensity of spill into the quarry associated with further erosion of the Main Breach. • Flow depths and intensities upstream from quarry continue to increase. • The Main Breach in the Western Levee has initiated.
	(g) 3:35pm • Commencement of outflow from the quarry through the south-eastern quadrant. • Breakout from Lockyer Creek at Quarry Access Road causeway commences with a rapid rise in local water levels. • Flow depths and intensities upstream from the quarry continue to increase.
	(h) 3:40 • Failure of Main Breach complete. • Breakout from Lockyer Creek at Quarry Access Road causeway continues with a rapid rise in local water levels. • Full breaking out of the right overbank flow path to the south of the quarry. • Flow depths and intensities upstream from the quarry continue to increase.

Figure 9.1 e to h – Chronology of Inundation, Quarry and Upstream 3:25pm to 3:40pm

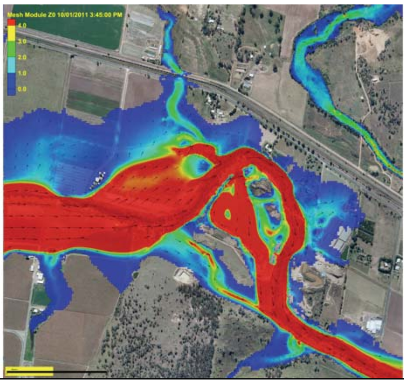
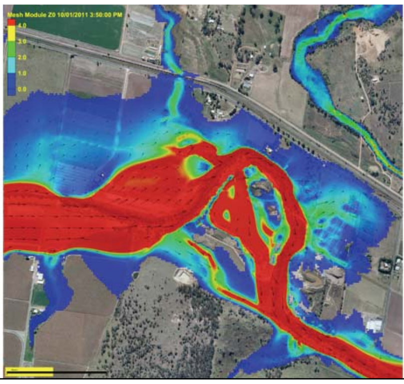
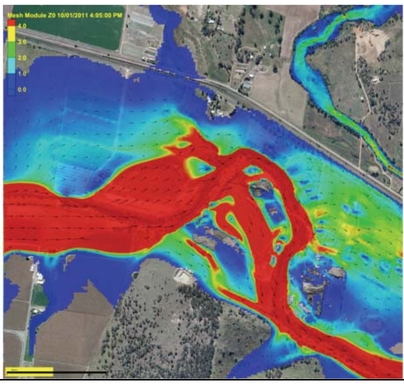
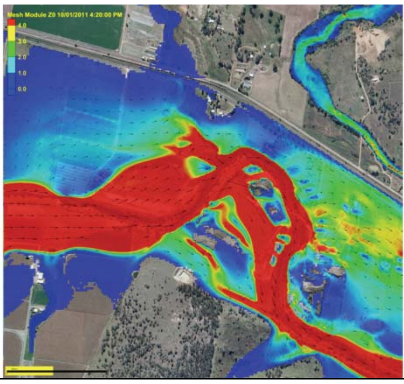
Image	Time and Commentary
	(i) 3:45pm • Breakout from Lockyer Creek at Quarry Access Road causeway continues with a rapid rise in local water levels. • Flow intensities starting to increase at the Quarry Access Road causeway breakout. • Full breaking out of the right overbank flow path to the south of the quarry. • Flow depths and intensities continue to increase throughout
	(j) 3:50pm • Initiation of erosion of the bunds in the Western Levee (other than in the area of the Main Breach). • Depth of water at 1615 Gatton-Helidon Road exceeds 0.3m (my estimated time of closure for evacuation purposes). • Flows commence along the Western Overbank flow path.
	(k) 4:05pm • Western Levee erosion now almost fully complete. • Floodplain upstream from quarry extends clear across to Gatton-Helidon Road.
	(l) 4:20pm • Flood levels close to peak.

Figure 9.1 i to l – Chronology of Inundation, Quarry and Upstream 3:45pm to 4:20pm

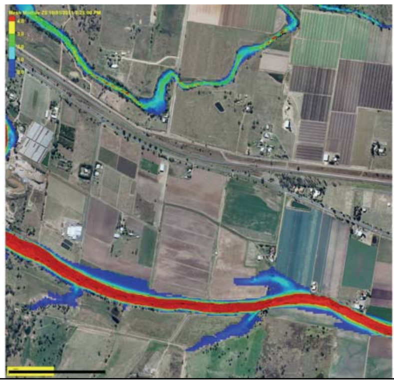
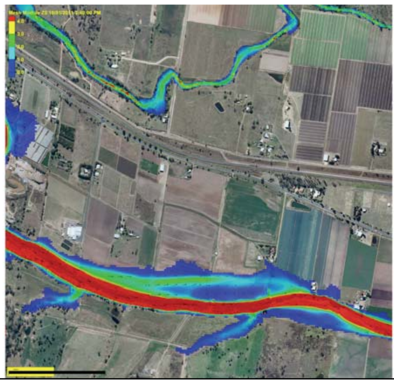
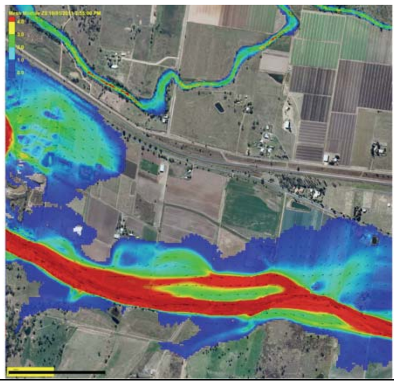
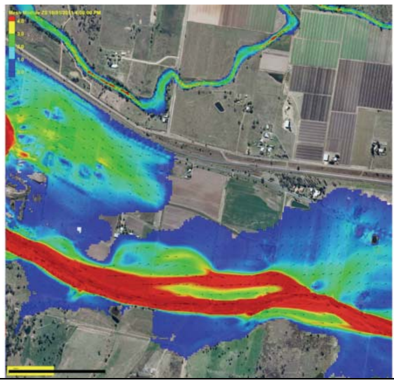
Image	Time and Commentary
	(a) 3:25pm • Flow depths begin to increase in Lockyer Creek and commence backing up in local drainage paths.
	(b) 3:40pm • Flow depths continue to increase. • Flow break out commences at the start of the South-Western Overbank path. • Note that erosion of the Main Breach has already been initiated at the quarry.
	(c) 3:55pm • Flow depths continue to increase. • South-Western Overbank flows have reached the Gatton-Helidon Road. • Western Overbank flows have commenced moving towards Grantham from their start at the quarry. • Note that Western Levee erosion now almost fully complete.
	(d) 4:00pm • South-Western Overbank flow depths have exceeded 0.3m (my estimated time of closure for evacuation purposes). • South-Western Overbank have commenced flowing through Western Grantham. • Western Overbank flows have continued moving towards Grantham.

Figure 9.2 a to d – Chronology of Inundation, Western Grantham 3:25pm to 3:40pm

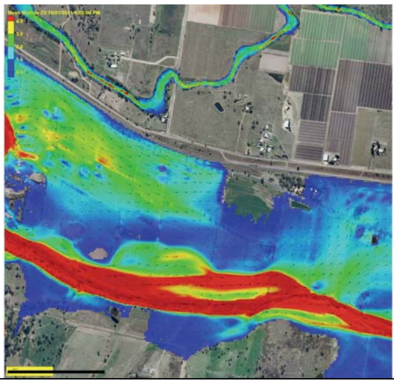
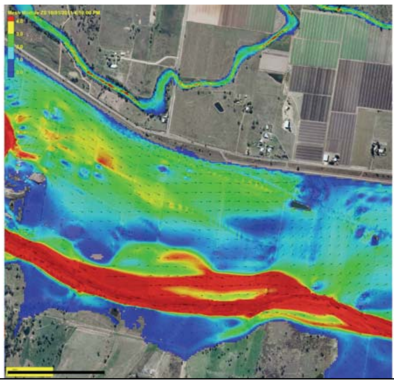
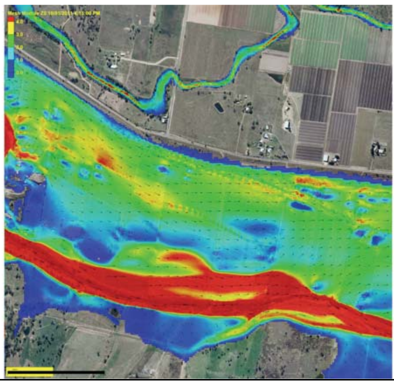
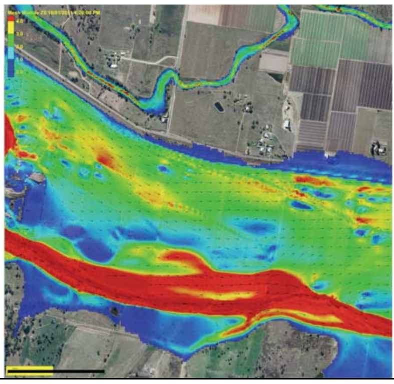
Image	Time and Commentary
	(e) 4:05pm • South-Western Overbank flows have further inundated Western Grantham. • Western Overbank flows now nearing Western Grantham.
	(f) 4:10pm • Western Overbank flows have now joined with the South-Western Overbank flows, and continue to travel through Western Grantham.
	(g) 4:15pm • Western and South-Western Overbank flow continue to travel through Western Grantham.
	(h) 4:20 • Floodwater has commenced overtopping the railway embankment.

Figure 9.2 e to h – Chronology of Inundation, Western Grantham 4:05pm to 4:20pm

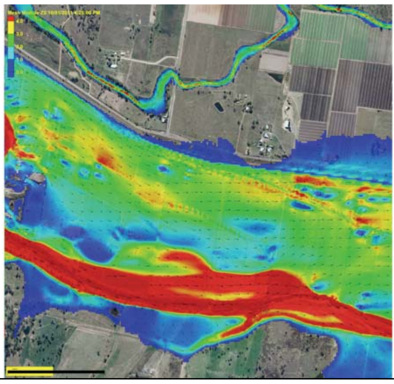
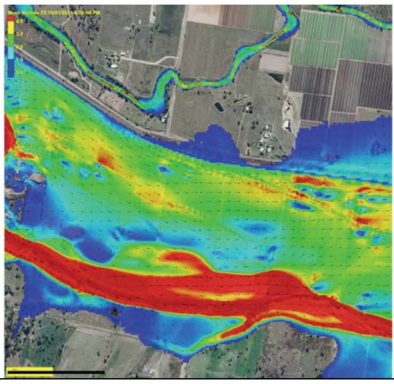
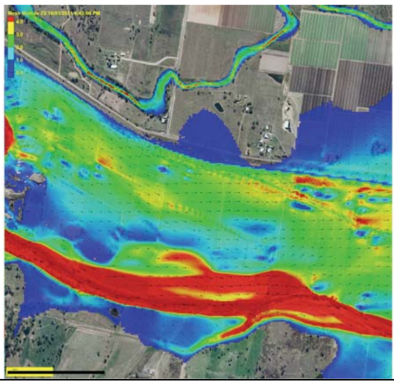
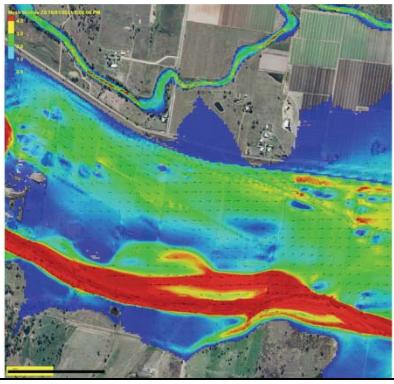
Image	Time and Commentary
	(i) 4:25pm • Flood flow intensities in Western Grantham appear to be around their peak.
	(j) 4:30pm • Flood flow intensities in western Grantham appear to be around their peak.
	(k) 4:45pm • Flood depths appear to be close to their peaks
	(l) 5:00pm • Flood depths appear to be close to their peaks

Figure 9.2 i to l – Chronology of Inundation, Western Grantham 4:25pm to 5:00pm

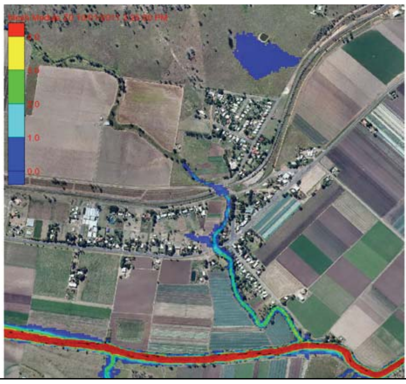
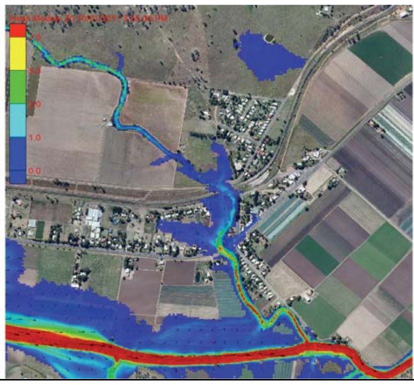
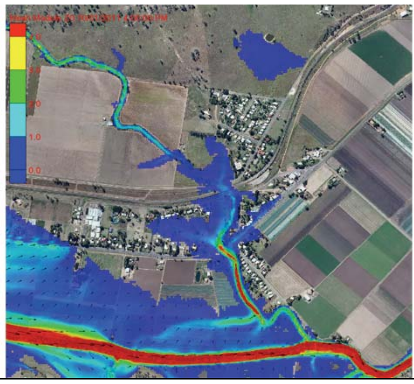
Image	Time and Commentary
	(a) 3:25pm • Sandy Creek has just surcharged and commenced inundating Central Grantham.
	(b) 3:40pm • Sandy Creek surcharge continues inundating Central Grantham. • Note that erosion of the Main Breach at the quarry has already been initiated.
	(c) 3:55pm • Lockyer Creek has broken its banks with inundating flows rapidly approaching Grantham from the South-Western Overbank flow path. • The Gatton-Helidon bridge over Sandy Creek has now been overtopped and within 1 minute the depth will exceed 0.3m (my estimated time of closure for evacuation purposes). This will result in closure of the last evacuation route for Western Grantham. • Note that erosion of the Main Breach has completed. • Note that erosion of the bunds in the Western Levee (other than in the area of the Main Breach) have commenced.
	(d) 4:00pm • Floodwater from Lockyer Creek over the South-Western Overbank flow path has overtopped the Gatton-Helidon Road in Western Grantham, but these floodwaters have not yet reached the road within Central Grantham. • Sandy Creek surcharge is continuing to inundate Central Grantham, but not as quickly as it is occurring in Western Grantham.

Figure 9.3 a to d – Chronology of Inundation, Central Grantham 3:25pm to 3:40pm

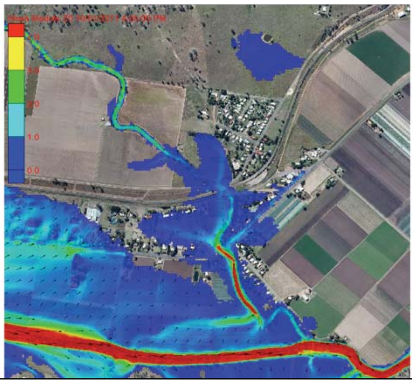
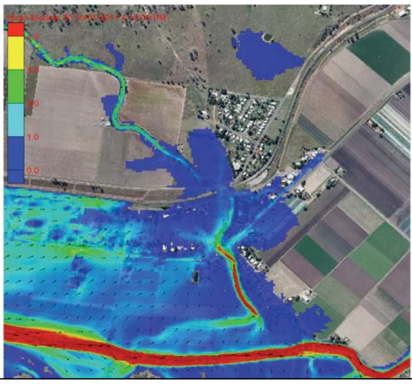
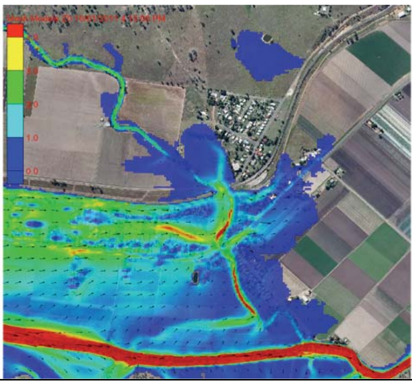
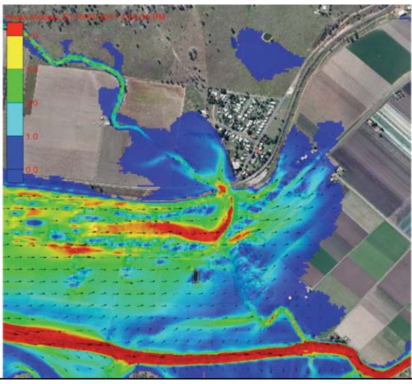
Image	Time and Commentary
	(e) 4:05pm • Floodwater from the Western Overbank flow continues to encroach on Grantham at speed, while Sandy Creek surcharge continues to rise.
	(f) 4:10pm • The remaining properties in the vicinity of Nicholls Street become inundated.
	(g) 4:15pm • Flood inflows under the rail bridge significantly intensify as the fast moving floodwater from the South-Western Overbank flows now reach this location. • Floodwater from the west continues through Central Grantham.
	(h) 4:20 • Floodwater has commenced overtopping the railway embankment. • Flood inflows under the rail bridge continue to intensify as the fast moving floodwater from the South-Western Overbank flows combine with those from the Western Overbank flows. • Flow intensities have increased in Eastern Grantham.

Figure 9.3 e to h – Chronology of Inundation, Central Grantham 4:05pm to 4:20pm

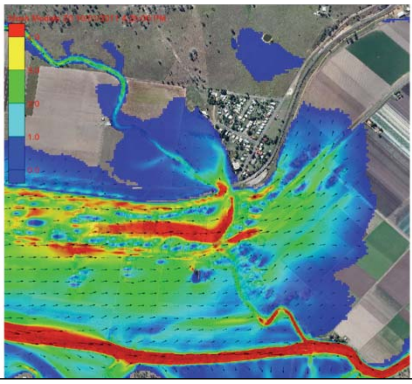
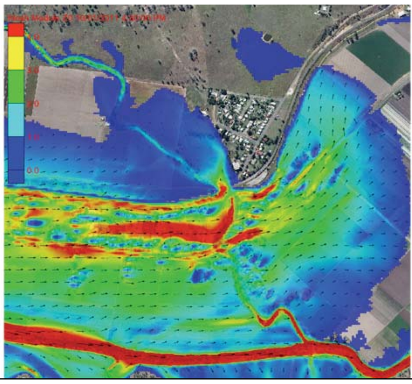
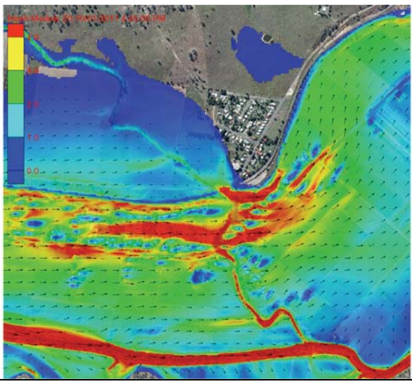
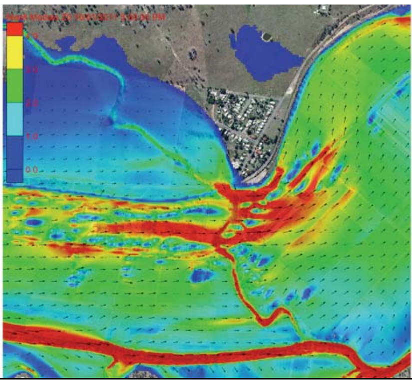
Image	Time and Commentary
	(i) 4:25pm • Floodwater continues to overtop the railway embankment. • Inflows under the rail bridge continue at high intensity. • Flow intensities have increased in Eastern Grantham.
	(j) 4:30pm • Floodwater continues to overtop the railway embankment. • Inflows under the rail bridge continue at high intensity. • Flow intensities continue to increase within Eastern Grantham.
	(k) 4:45pm • Flows at the rail bridge location reverse in consequence of rail embankment overflows. • Flow intensities in Eastern Grantham significantly increase in conjunction with the rail bridge flow reversal. • Flood flow intensities in Central and Eastern Grantham around their peak.
	(l) 5:00pm • Flood flow intensities in Central and Eastern Grantham around their peak.

Figure 9.3 i to l – Chronology of Inundation, Eastern Grantham 4:25pm to 5:00pm

10 Effect of the Grantham Quarry on Flooding

10.1 General Arrangement

308. Grantham Quarry is located on a sharp meander, or "hairpin bend", of Lockyer Creek located approximately 3km upstream of Grantham. An extract from an aerial photograph taken in 1982 is shown in Figure 10.1 (QAP4014-13, DERM) and shows the quarry area in its very early stages of development.

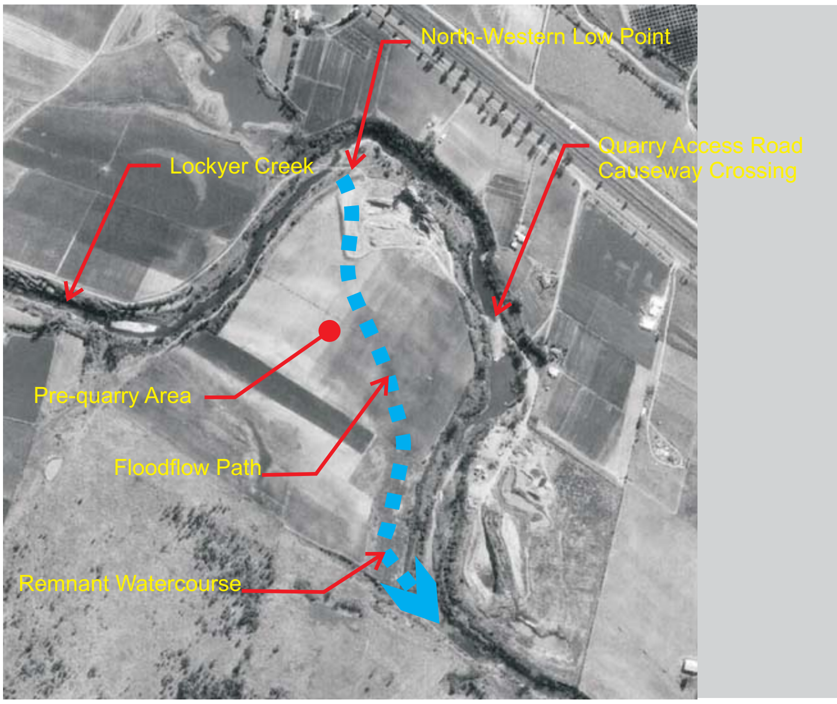


Figure 10.1 – Quarry Area in 1982 Overlaid with Flood Path

309. From the work undertaken by Mr Starr (Geotechnical Expert), I see that the natural surface along the creek bank on the western side of the pit sloped to a low point (level 122mAHD) at the location of the large Main Breach (Figure 10.3) that developed during the course of the 10th January 2011 flood event. The reconstructed likely pre-quarry surface contours shown by Mr Starr in Plate 21 of his report (also refer to Appendix B.7) indicates the existence of a slightly depressed topography running between this low point (north-western side of the quarry area) and what appears to be the remnant of an old watercourse in the south-east of the quarry area.

310. The pre-quarry topography defined by Mr Starr indicates to me that before the quarry was constructed there would be occasion when floodwater would have broken out of the right bank of the creek at the low point on the bank at north-western corner of the hairpin, and flowed overland to re-join Lockyer Creek in the south-eastern quadrant. I have annotated Figure 10.1 to show this.

311. The current quarry comprises a pit, over 12m deep and with a surface area of about 10ha. At the time prior to the 10th January 2011 flood event it was surrounded by a series of bunds and levee banks of varying crest levels that afford it some additional protection against flood inundation, as indicated in Figure 10.2 (August 2010 from LVRC).

Figure 10.2 – Oblique Showing Pre-flood Quarry

312. The quarry experienced significant damage as a result of the flood, and in particular significant erosion to the bunds on the Western Levee and breaching of the natural creek bank at the location of the north-western low point, as indicated in Figure 10.3 (January 2011 from LVRC).

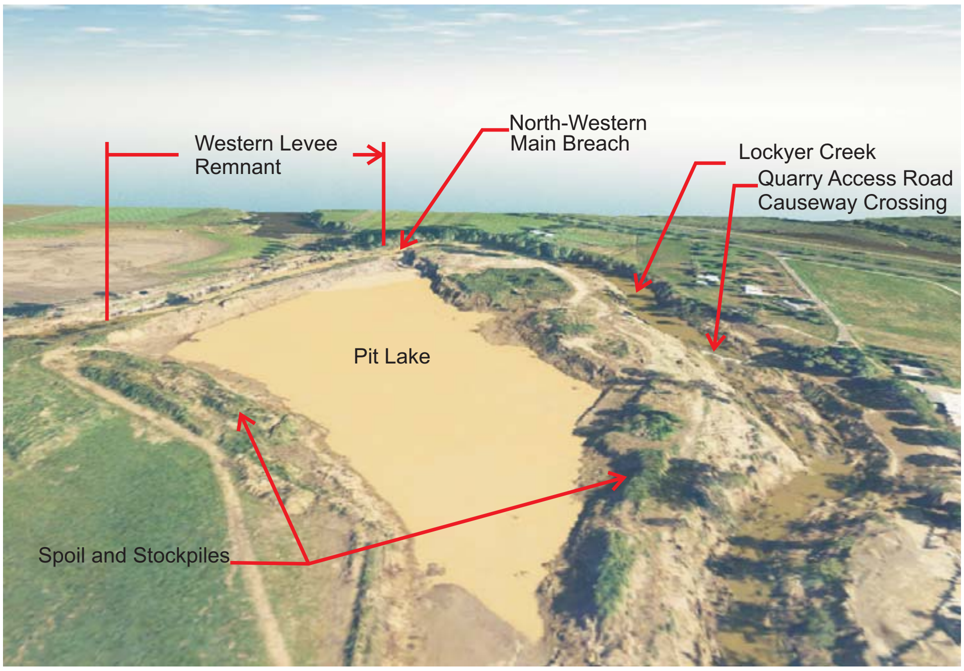


Figure 10.3 – Oblique Showing Post-flood Quarry

10.2 Overview of Assessment

313. I have needed to consider a number of alternative scenarios in determining the likely effect of the quarry on the flooding of Grantham:

- No Quarry – conditions before commencement of any quarry workings, as indicated in Figure 10.1, but without any quarry workings at all;
- Most Likely – my considered most likely quarry configuration and status immediately prior to the 10th January 2011 flood; and
- Worst Case – my considered quarry configuration and status, immediately prior to the 10th January 2011 flood, that would be likely to produce the greatest impact on flooding in Grantham.

314. I have considered a range of sensitivity cases within each of the Most Likely and Worst Case configurations to assist me in determining the significance of uncertainties associated with the status of the Western Levee prior to the flood, the time at which the levee started to fail, and the duration that the levee took to fail.

315. I note that I had already undertaken a number of other sensitivity tests on the GFCOI model as part of its original setup. These are discussed in Section 12.

316. In my assessment, I have used the No Quarry scenario as a point of reference against which I have compared the other scenarios.

317. I have presented a discussion outlining my parameterisation of these scenarios in Section 10.3 below. Outcomes from my assessment then follow: No Quarry assessment in Section 10.4; Most Likely scenario assessment in Section 10.5; and Worst Case scenario assessment in Section 10.6. My considerations and conclusion are presented in Section 10.7.

10.3 Method Parameters

General

318. The effect of the quarry pit and levees on flood flows and levels is the result of an array of individual hydraulic mechanisms, all working in concert, but with different cause and effect actions. It is a highly dynamic process and the net result is in consequence of how these mechanisms combine; their sequence, their magnitude, and their timing, all cast against the background of the rapidly increasing flows down Lockyer Creek.

319. I have obtained relatively good information defining the condition (topography) of the quarry and levees before and after the 10th January 2011 flood. However, I have found little information that I can use to directly quantify the actual sequence of levee failure. Although my experience has led me to conclude upon what I have called the “Most Likely” levee failure scenario, uncertainty still remains.

320. As I have stated previously, the primary purpose of the GFCOI model is to provide me with a tool that I can use to best understand the circumstances of the flood, its impacts on Grantham and surrounds, and the likely impacts of man made works such as the quarry and the rail embankment.

321. To undertake my assessment I have run a series of separate simulation analyses (scenarios), each representing an alternative possible configuration of the levee failure sequence. I have

designed these scenarios to provide reasonable cover of those items of uncertainty that I consider to be of key significance to the purpose of my investigations:

- the time that Western Levee breaching commenced;
- the duration over which the Western Levee breaching occurred; and
- the level of water within the quarry pit (pit lake) when the Western Levee breaching commenced.

No Quarry Scenario

322. I have used the "No Quarry Scenario" as a point of reference against which to compare the other scenarios.

323. The set up of the model for this scenario has been to represent the topography of the quarry site as it would have been prior to any quarrying operations. I have achieved this by removing the definition of the quarry pit, levees and associated surface disturbances (such as spoil piles, stockpiles, and roads associated with quarry operations) and replacing it with topography defined by Mr Starr (Geotechnical Expert). Details of this modified "pre quarry" topography is presented in Appendix B.7.

324. The main effects that this has had on the simulation model has been to:

- remove the flow restriction introduced by the presence of the Western Levee;
- remove the flood storage "buffer" that serves to absorb floodwater when it flows into the pit; and
- re-establish the time that it takes overland flow traversing the quarry pit area to what it would have been pre-quarry.

Most Likely Scenario

325. The "Most Likely" scenario is based directly on the GFCOI model (Section 8). In summary the scope of this scenario is as follows:

- pit lake levels at 120mAHD (2m below full pit capacity); and
- Eastern and Western Bunds overtopped prior to initiation of breaching; and
- progressive breach initiation in the form modelled in the LVRC hydraulic model.

326. The variant I have considered in the Most Likely scenario is the duration of failure, once breaching has initiated. The range of failure durations that I have considered are:

- fast – 5 second failure duration;
- typical – 10 minute failure duration; and
- slow – 1 hour failure duration.

Worst Case Scenario

327. I have designed a set of scenarios specifically to determine if it is theoretically possible to configure the quarry so that it produces a "dam failure" (or Worst Case) type flood wave that would propagate downstream to Grantham. To be of any significance to flood impacts at

Grantham this wave would need to travel the Western Overbank flow path from the hairpin bend at the quarry.

328. My definition of this "Worst Case" type condition was as follows:

- I have assumed that the pit lake is high to overflowing before the flood event started; my most conservative estimate puts it at 122mAHD or at pit capacity full; and
- I have assumed that the full Western Levee of the quarry extends above flood level and is never overtopped; failure is instead artificially triggered when water levels adjacent to the Quarry Access Road causeway get to a range of predefined levels; and
- the duration of failure of breach is very fast (duration 5 seconds) – that is, like a dam wall falling over.

329. The triggering mechanism I used produced a range of failure initiation times extending from 3:13pm to 4:16pm.

<u>Reference Locations</u>

330. I have established the likely effects on flooding in Grantham using two indicator methods:

- comparison of time-series of simulated flood flow rates, at the downstream locations marked in Figure 10.4; and
- comparison of depth and flow intensity hydrographs, at the locations marked in Figure 10.5.

Figure 10.4 – Downstream Reporting Locations

Figure 10.5 – Flow Depth and Intensity Hydrograph Locations

331. I have selected the downstream reporting locations marked in Figure 10.4 because:

- Lockyer Creek – flows past this location affect all flooding that emanates from the downstream Lockyer Creek, including the South-Western Overbank flow and Sandy Creek back flows;
- Western Overbank – flows past this location lead directly to Western Grantham from the west;
- South-Western Overbank – flows past this location represent the initial breakout front that first reaches Western Grantham from the south-west.

332. I have selected the flow hydrograph reporting locations indicated in Figure 10.5 because they give representative coverage of depth and flow intensity measurements within and around Grantham. Depth and flow intensity measurements are good indicators for the quantification of flood hazard.

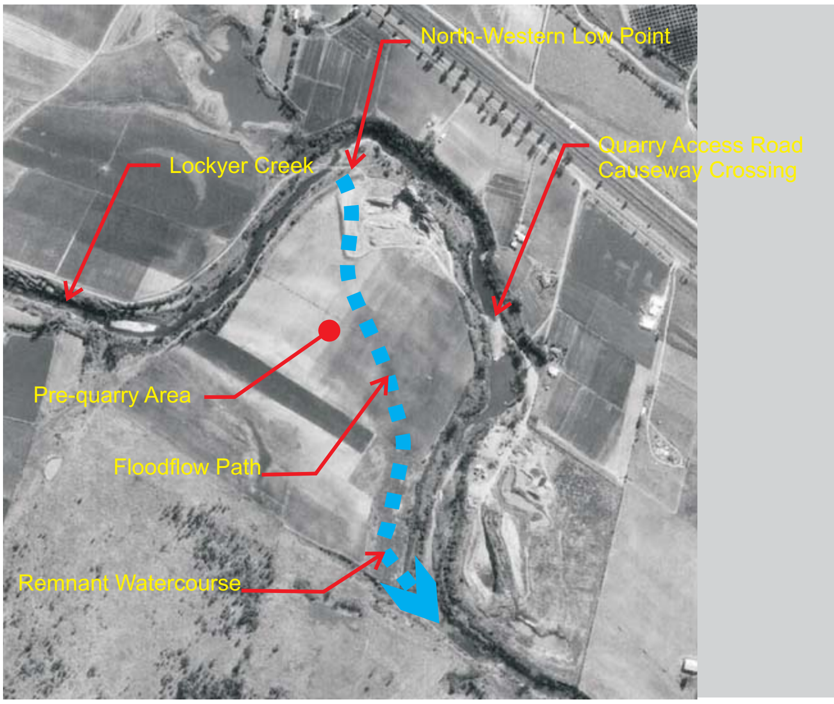
333. Further, any changes to flood flow hydrographs will translate downstream to Grantham. Although the effect on flooding is not a linear relationship, if there is an increase in flow then there will be an increase in flooding, and vice versa. The other aspect of importance is timing, which can be interpreted in the same manner as flow rate.

10.4 No Quarry Scenario Assessment

334. The pre-quarry surface that I have used was based on the outcomes of field investigations and interpretations undertaken by Mr Starr (Geotechnical Expert). I have developed a model of the surface topography that would likely to have been at the location of the quarry. I then used this new surface (pre-quarry) as a replacement for the “pre-flood” with quarry surface (August 2010) that was used by Jacobs in the original development of the LVRC hydraulic model (and which I have then used to construct the GFCOI model).

335. I have presented a discussion of No Quarry (pre-quarry) conditions in Section 10.1 and provided more technical details in Appendix B.7. Key aspects are reproduced in Figure 10.6 below (a larger version is contained in Appendix C).

(a) Grantham Quarry Area (circa 1982)



(b) No Quarry (pre-quarry topography)

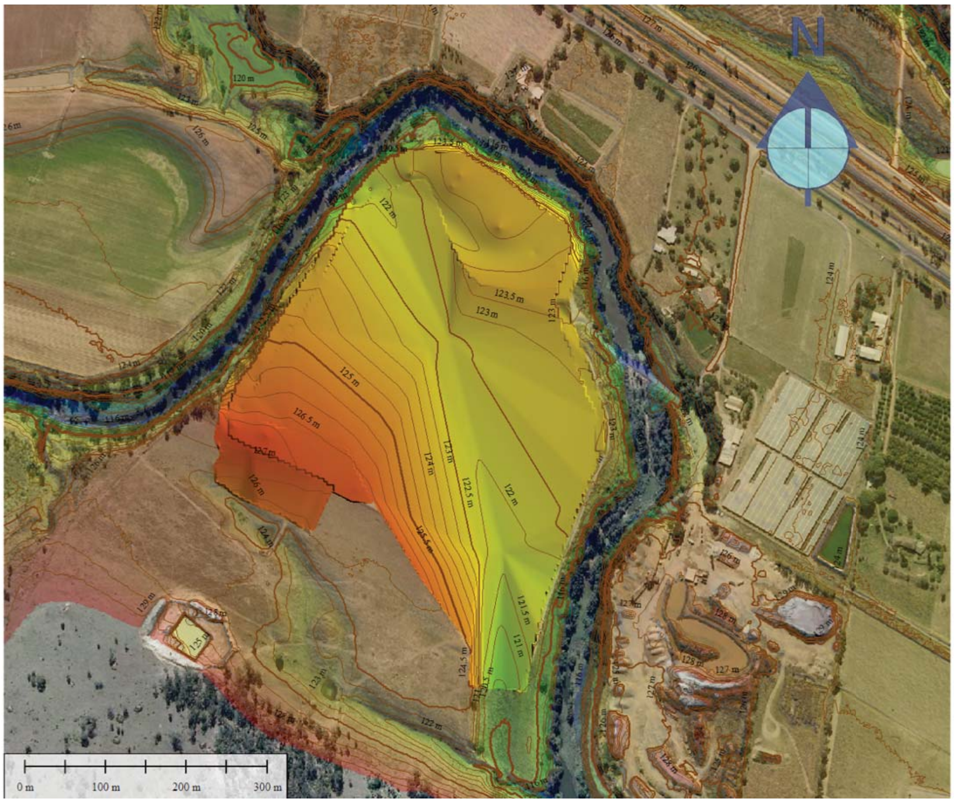


Figure 10.6 – Surface Topography at the Grantham Quarry Site

336. As indicated in Figure 10.6, under pre-quarry conditions it is expected that breakout flood flows would have commenced at a low spot in the north-western quadrant of the area (the same location where the Main Breach developed during the 10th January 2011 event). These flows would have re-joined Lockyer Creek in the south-eastern quadrant via what appears to me to be a remnant depression from an ancient water course.

337. As indicated by the topography contours in Figure 10.6, the initial overland flow path follows a shallow depression from north-west to south-east. There are no obstructions to flow in the pre-quarry area.

338. Applying the pre-quarry topography to the GFCOI model has produced simulated hydrographs at the downstream reporting locations identified in Figure 10.4 and the depth and flow intensity locations marked in Figure 10.5.

Figure 10.7 – No Quarry Reference Downstream Flow Location

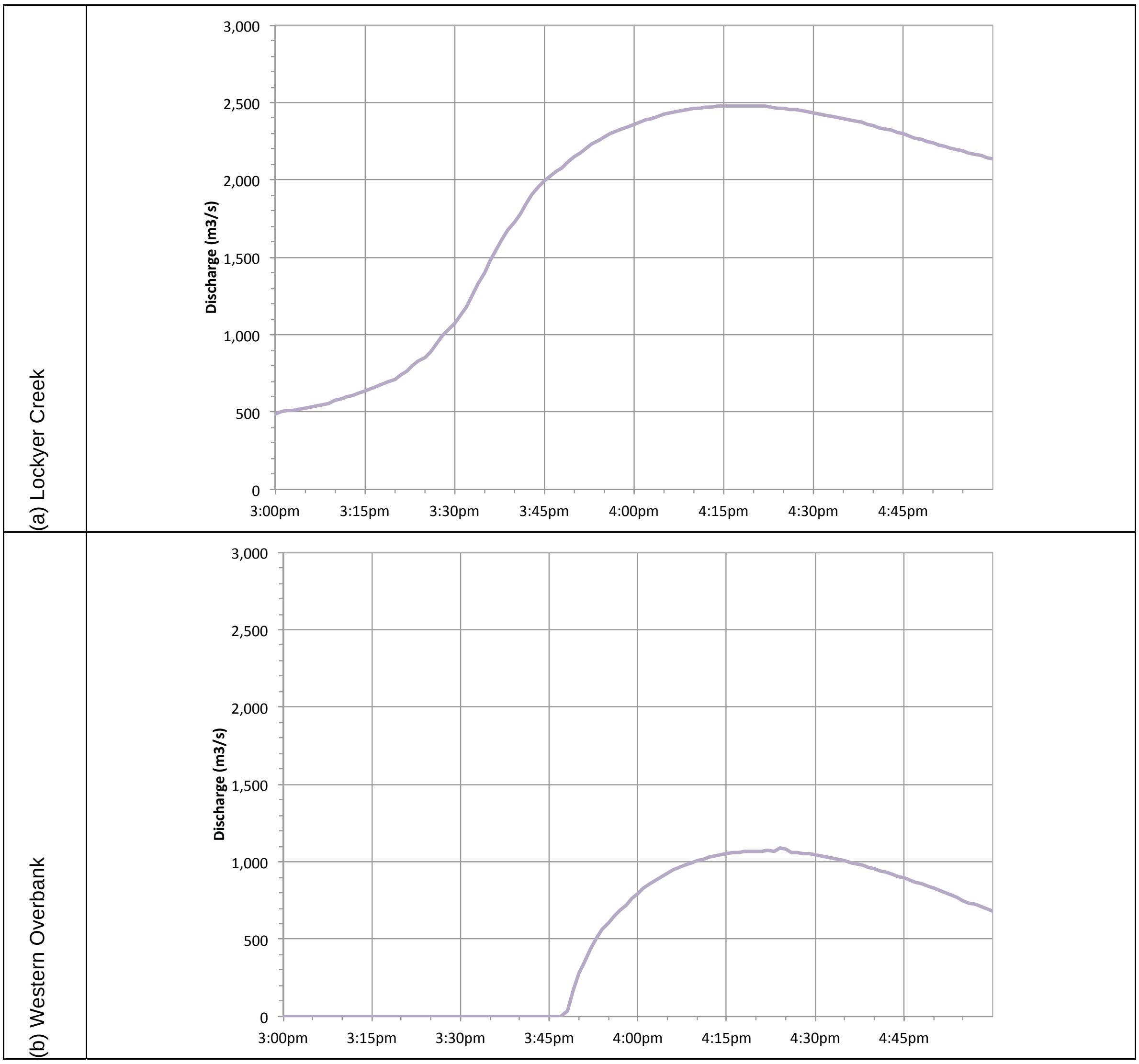


Figure 10.7 (continued) – No Quarry Reference Downstream Flow Location

(c) South-Western Overbank

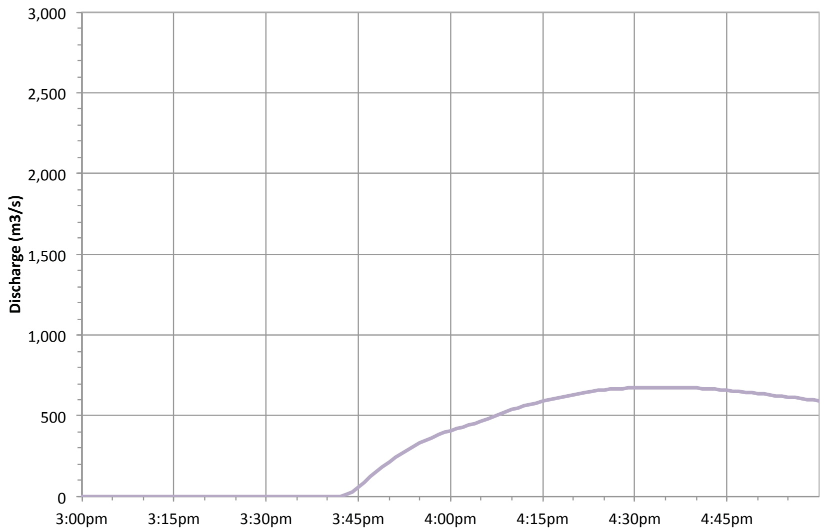


Figure 10.8 – No Quarry Reference Flow Depth and Intensity Location

(a) Quarry Access Road

(b) 1414 Gatton-Helidon Road

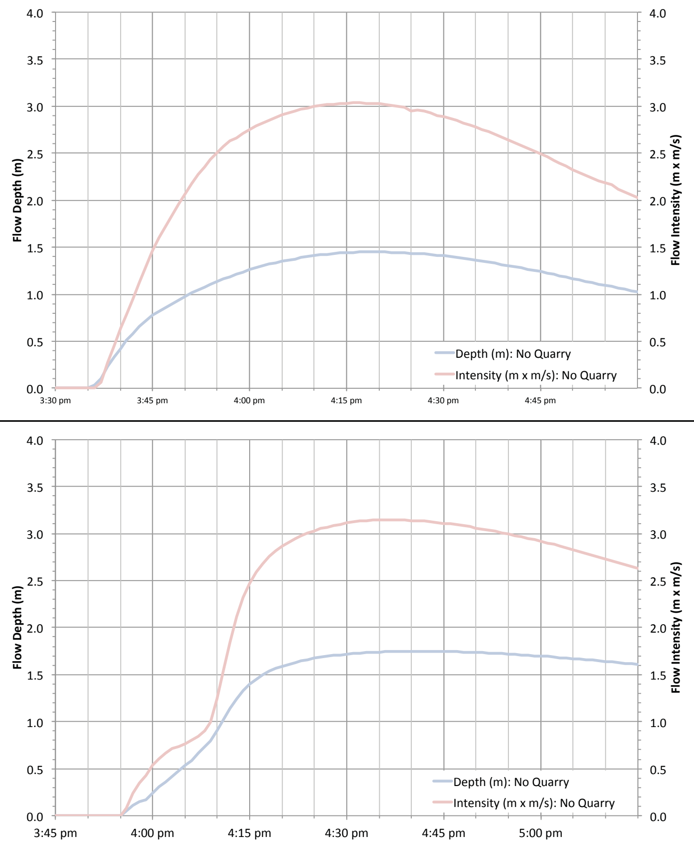
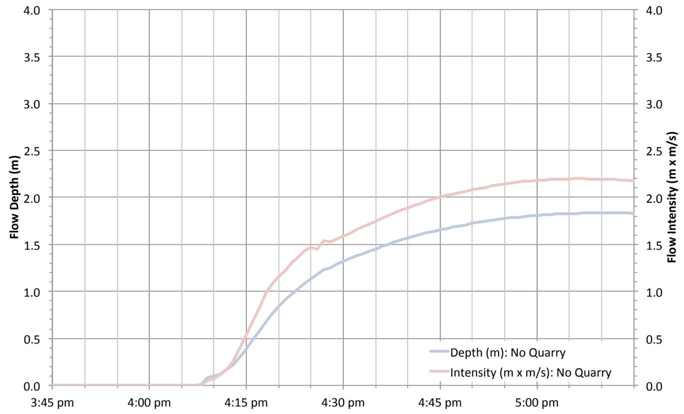
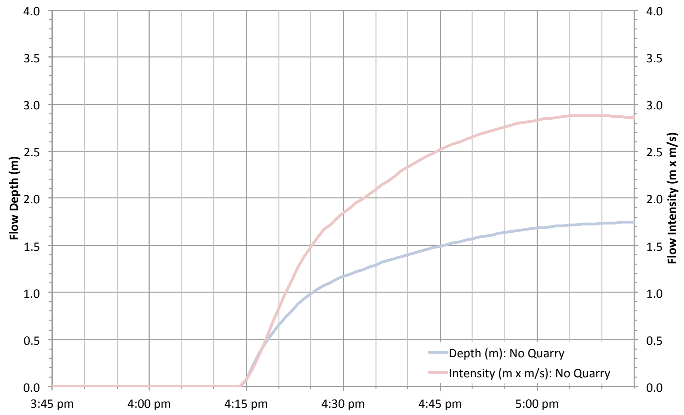


Figure 10.8 (continued) – No Quarry Reference Flow Depth and Intensity Location

(c) 12 Anzac Avenue



(d) 26 Anzac Avenue



339. My interpretation of these simulation hydrographs is summarized as follows:

- the Western Overbank flood flow path remains active under No Quarry conditions;
- breakout of flows from Lockyer Creek at the hairpin bend near Quarry Access Road commence at about 3:34pm;
- inundation of Western Grantham at 1414 Gatton-Helidon Road commences at about 3:54pm with the arrival of breakout flow from Lockyer Creek to the south-west;
- the Western Overbank breakout flows reach Western Grantham at 1414 Gatton-Helidon Road at about 4:09pm, or about 15 minutes after the initial inundation front;
- inundation of Central and Eastern Grantham, as represented by flow characteristics at 12 Anzac Avenue and 26 Anzac Avenue, commence at about 4:06pm and 4:14pm respectively;
- the rate of rise of flow depth and intensity in Western Grantham (1414 Gatton-Helidon Road) is significantly more than that at Central and Eastern Grantham; and
- at all locations the flow intensity increases rapidly from zero to above 0.5m^2/s in about 6 minutes.

10.5 Most Likely Scenario Assessment

340. I have chosen the Quarry levee configuration provided in the LVRC models as a reasonable representation of the Most Likely Scenario for the manner in which it describes the Western Levee breach mechanism. For the reasons explained in Section 8.6 (Modelling the Western

Levee Failure), I consider the method employed by Jacobs in the LVRC models to model the failure mechanism to be suitable and accordingly I have used this method for the most likely scenario. I have presented details of this configuration in Section 8 of my report and Appendix B.6.

341. Although eye-witness accounts refer to the time when initial inflows into the pit occurred (Mr Sippel's statement 2015 and transcript 2015) there is no direct observation that provides insight into the onset and rate of levee erosion. The only hard data available is topographical survey of the eroded surface post-flood.

342. As a consequence I have had to test the likely effect of levee erosion time on simulated flood characteristics by considering a range of erosion times and using the GFCOI model to quantify the expected effects on downstream flooding.

343. I have considered three scenarios with erosion times as follows:

- 5 seconds (i.e. fast);
- 10 minutes (i.e. typical); and
- 60 minutes (i.e. slow).

344. For the purposes of this assessment I have assumed that both the Eastern and Western Bunds, and Main Breach develop to their post-flood extent over the erosion time. I have also assumed that the erosion of the Eastern and Western Bunds, and Main Breach is triggered at three points (A, B, C) as indicated in Figure 10.9 below. I have associated these triggers to individual bunds (shown as Levee 1 to 5) also shown in Figure 10.9. When flood levels reach the same height as that of a trigger it will cause the start of simulated erosion of the associated bund. For example, when the water level reaches the trigger level for location C simulated erosion will then commence at the Main Breach. I have provided in Appendix B.6 a table that specifies the flood level for each trigger location and the corresponding time of trigger. Due to the difference in heights at locations A, B and C triggering will occur at different times and bund erosion will occur progressively.

Figure 10.9 – Quarry Failure Trigger Level Locations

345. The trigger heights have also been set so as not to trigger until the levees as depicted in Figure 10.9 above have been overtopped. I have done this in consideration of the rapid rate of rise of the flow hydrograph, which I consider would most likely result in the occurrence of a top down erosion breach of the Western Levee.

346. The relevant flood levels that trigger the Main Breach and the remainder of the Eastern and Western Bunds are broadly:

- Main Breach: 125.0mAHD
- Eastern and Western Bunds: 127.6mAHD

347. Applying the trigger mechanism described above to the GFCOI model produces failure initiation times. I have extracted these times from simulation outcomes as well as the status of flood levels in Lockyer Creek and the pit lake at the same time. This information is listed in Table 10.1.

Table 10.1 – Most Likely Scenario Trigger Status

Failure Duration Scenario	Time of Levee Failure (hh:mm)	Upstream Water Level (mAHD)	Water Level in Pit Lake (mAHD)	Creek to Pit Drop Height (m)
Main Breach	3:25 pm	125.0	120.2	4.8
Eastern and Western Bunds	3:45 pm to 3:56 pm	127.6	125.6 to 126.4	2.0 to 1.2

348. I have then used the GFCOI model to produce simulated hydrographs at the downstream reporting locations identified in Figure 10.4 and the depth and flow intensity locations marked in Figure 10.5. Those hydrographs are indicated in Figures 10.10 and 10.11 below.

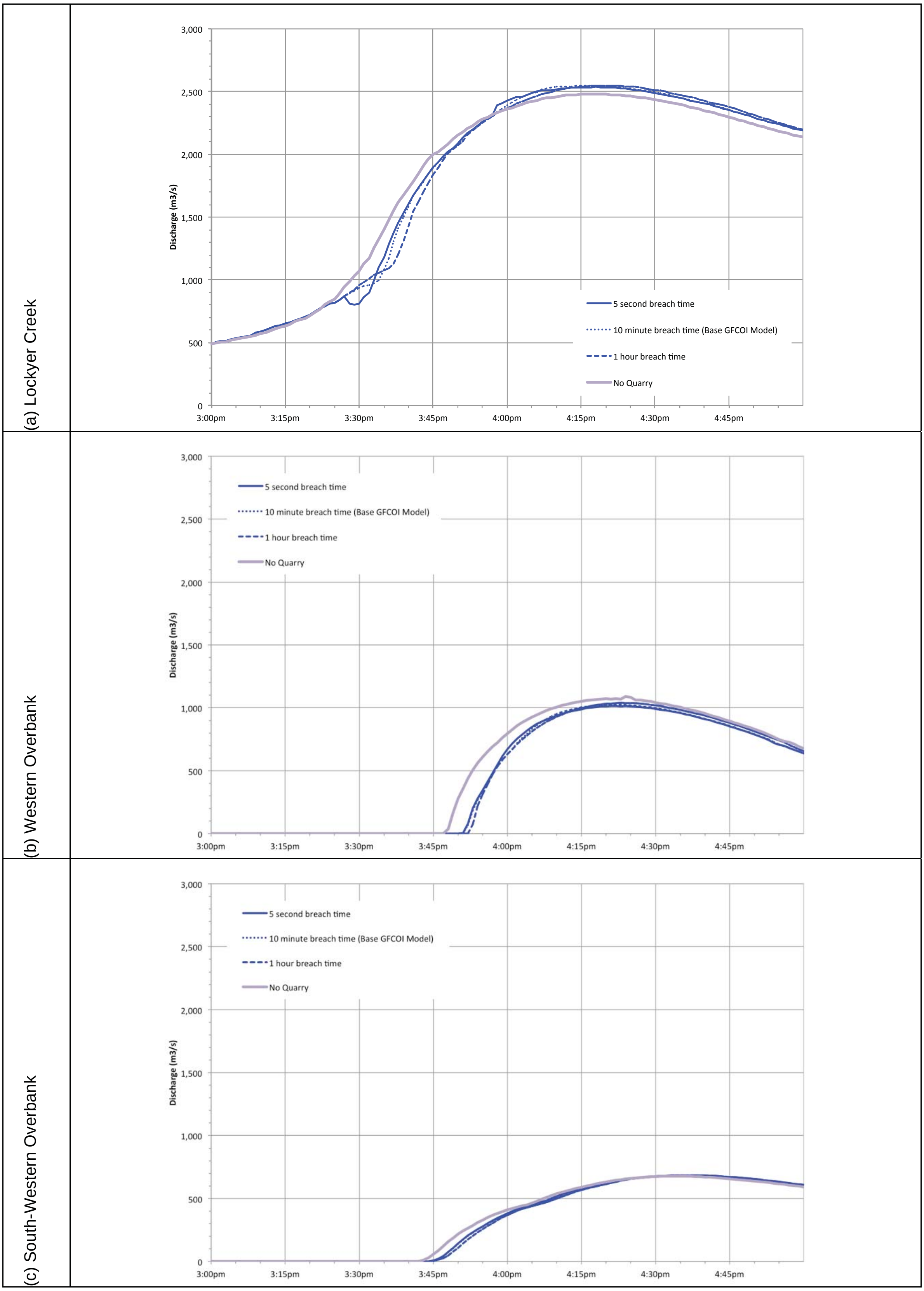
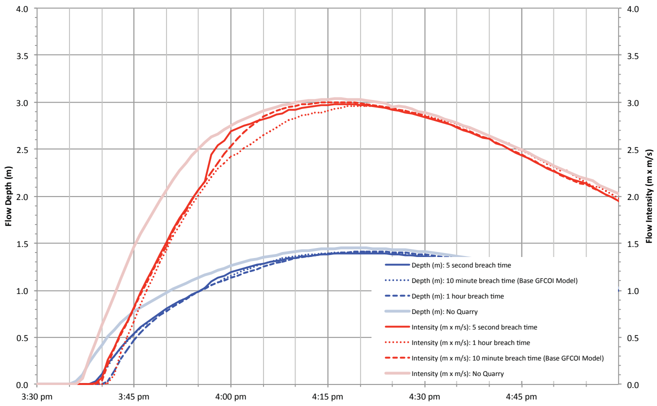
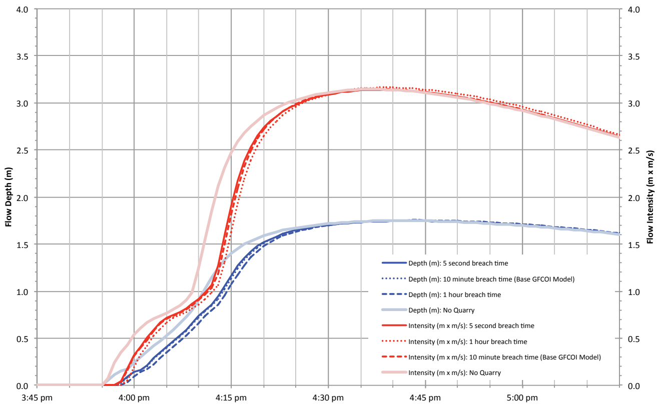


Figure 10.10 – Effect of Most Likely Quarry Levee Failure on Downstream Flow

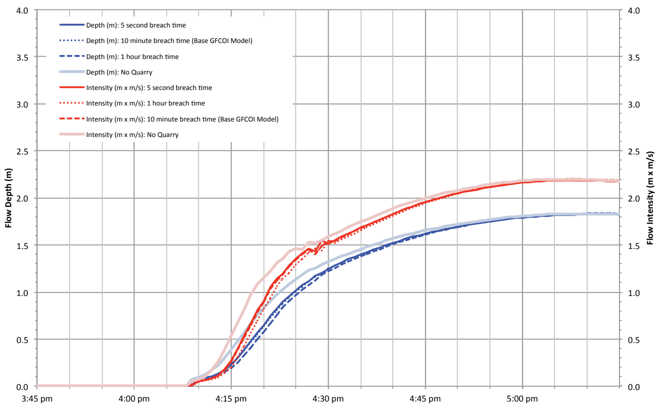
(a) Quarry Access Road



(b) 1414 Gatton-Helidon Road



(c) 12 Anzac Avenue



(d) 26 Anzac Avenue

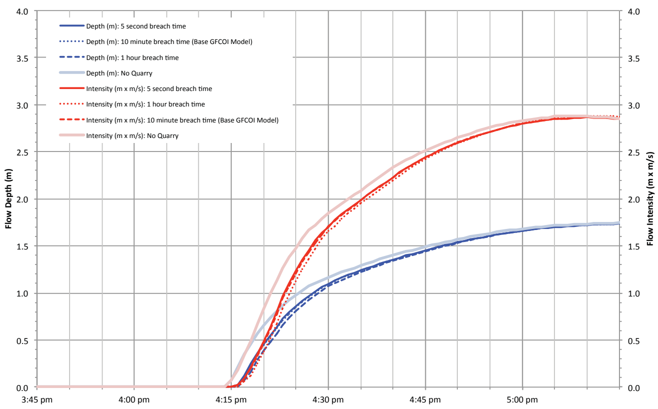


Figure 10.11 – Effect of Most Likely Quarry Levee Failure on Flow Depth and Intensity

349. My interpretation of these hydrographs is summarized as follows:

- the Grantham Quarry is likely to have delayed the flow hydrographs by about 1 to 3 minutes;
- adjacent to the quarry, at Quarry Access Road, the shorter the duration of levee failure the closer flooding characteristics approach no quarry conditions;
- further afield in Eastern, Central and Western Grantham flooding characteristics appear to be relatively insensitive to duration of levee failure;
- peak flood flows in Lockyer Creek are seen to be slightly raised compared to the no quarry scenario, (this being because after levee breaching the flood flow path through the quarry provided less resistance to flow than what would be likely for flow over the pasture assumed to be present in the no quarry case);
- peak overbank flow rates with the quarry are similar or lower than the corresponding flow rates in the no quarry case;
- the presence of the quarry does not result in any increase in overall flow intensity in Grantham compared to the no quarry case.

350. Given the little difference between the 5 seconds, 10 minutes and 1 hour duration of failure in terms of downstream response, I have given consideration to estimating the most likely duration of levee failure through erosion, after initiation. The method I have used has been to first obtain an estimate of the volume of material that I would expect to have been removed from this atypical erosion mechanism (top down erosion breach). I then applied the outcome of research by Froehlich (2008), which provides an empirical method (based on actual historical records of a range of breach embankment failure times for dams and levees). The method equates failure time to a function of breach volume and breach depth, assuming an idealized trapezoidal section of breach. It is a generalised method that essentially provides a "best fit" approach to the data upon which it was based. I have applied this method so that I could estimate the duration time of the breaches to the Western Levee.

351. I have obtained calculations from Mr Starr (Geotechnical Expert) of the volume of material lost from the Western Levee during the course of the 10th January 2011 flood. The outcome of Mr Starr's calculation is shown in Figure 10.12 below and shows:

- Western Level, Main Breach: net 11,740m^3; and
- Western Embankment (Eastern and Western Bunds): net 19,140m$^{3.}$

Figure 10.12 – Pre and Post Flood Volume Comparisons (Starr 2015)

352. The cross-sections that I have plotted in Figure 8.16 and 8.17 (Section 8.8.2) show the difference between pre and post flood surface levels along the centreline of the Western and Eastern Bunds, and at the Main Breach, in the Western Levee. From these I have observed that:

- the Main Breach depth of scour was about 7.5m;
- for the remainder of the Western Levee;

- the Eastern Bunds were higher than the Western Bunds; and
- given this the highest scour depth in the area was about 3.4m (on average), being the scour depth for the Eastern Bund.

353. Applying the net volumes removed and associated observed scour depths to Froelich's method produced failure duration estimates of:

- Main Breach: 5 minutes; and
- Eastern / Western Bunds: 14 minutes.

354. I have interpreted the results to conclude that a reasonable estimate of duration of failure is between 5 and 14 minutes, from which I have adopted the average, rounded to 10 minutes.

355. On this basis I have concluded that the 10 minute failure duration is most likely and, accordingly, I have selected this duration for application to the most likely scenario to be used as the base GFCOI model.

10.6 Worst Case Scenario Assessment

356. In creating the Worst Case scenario set, my objective was to maximise the potential for the production of a detrimental surge of flood flows that would propagate downstream. I considered that this would be best met as follows:

- maximize the volume of any surging floodwater along the Western Overbank flow path towards Grantham;
- maximize the differential in water level between the creek adjacent to the Western Levee and pit lake;
- set the levee to fail rapidly and all in unison; and
- minimize the capacity of the pit area to absorb inflowing water from a levee breach.

357. Some of these objectives are interdependent to some extent. For example, minimising the absorption of inflowing water into the pit would require the setting of raised pit lake levels in the simulation, but this in turn would reduce the height differential between the ponded water behind the Western Levee and the pit (which I want to maximise). Also, delaying the simulated initiation of breaching of the levee, so as to accumulate more flood storage behind the Western Levee, would provide more opportunity for floodwater to backflow and fill the pit from Lockyer Creek in the south-eastern quadrant of the pit. This in turn would result in reduced water level differences between the ponded water behind the Western Levee and the pit (which I want to maximise).

358. I have therefore configured the GFCOI model to run a range of levee failure trigger conditions and then processed the results to produce an envelope of Worst Case simulation outcomes. My method of analysis was as follows:

- start the simulation with the height of the entire Western Levee to above peak flood level;
- start the simulation with the water level within the pit set to 122.0mRL; the level at which the pit lake would just start to free outflow from its south eastern corner (i.e. no in-pit flood storage capacity);
- set all the breaches in the Western Levee to all fail simultaneously, over a period of 5 seconds when triggered; and

- set the levee failure trigger to the time at which the water level in Lockyer Creek, at the location of the access causeway (east of the quarry), is simulated to reach a predefined flood level.

359. I have configured initiation of levee failure using the simulated water level in Lockyer Creek at the location marked Trigger D in Figure 10.13 below. Also, I have used the location marked Trigger B for water level reporting purposes (not to be confused with Trigger D).

Figure 10.13 – Quarry Failure Trigger Level Locations

360. Figure 10.14 presents a cross-section of the Lockyer Creek waterway at the causeway crossing. The cross-section shows the creek, the banks and the crown of Quarry Road. My selection of water trigger levels is also marked.

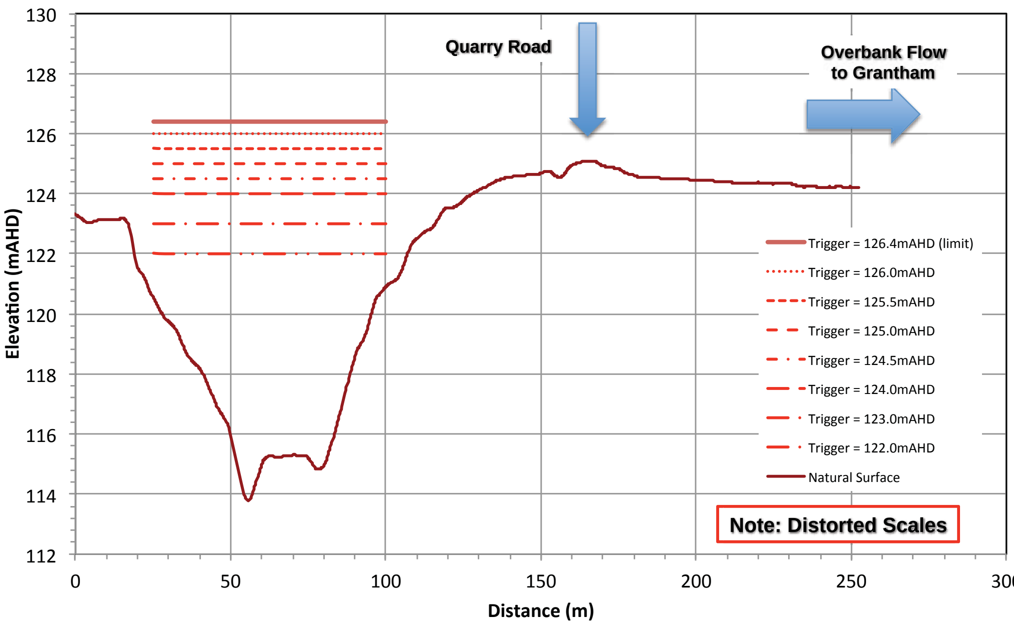


Figure 10.14 – Worst Case Trigger Levels: Quarry Causeway

361. I have extracted the simulated times of levee failure triggering, and the associated water levels in Lockyer Creek to the west of the levees (I have used the location marked as Trigger B, Figure 10.13 for reporting water level), from the GFCOI model for the trigger level scenario cases shown in Figure 10.13, as listed in Table 10.2.

Table 10.2 – Worst Case Trigger Status

Trigger Level Scenario (ref. location Trigger D)	Time of Levee Failure (hh:mm)	Upstream Water Level (mAHD) (at location Trigger B)	Water Level in Pit Lake (mAHD)	Creek to Pit Drop Height (m)
Trigger = 126.4mAHD	4:16 pm	128.5	126.4	2.1
Trigger = 126.0mAHD	3:52 pm	128.1	125.9	2.2
Trigger = 125.5mAHD	3:44 pm	127.8	125.3	2.5
Trigger = 125.0mAHD	3:40 pm	127.6	124.7	2.9
Trigger = 124.5mAHD	3:35 pm	127.1	123.4	3.7
Trigger = 124.0mAHD	3:30 pm	126.1	123.0	3.1
Trigger = 123.0mAHD	3:24 pm	124.6	123.3	1.3
Trigger = 122.0mAHD	3:13 pm	123.4	122.8	0.6

362. The simulation outcomes listed in Table 10.2 show that as the trigger level heights increase over the range 122.0mAHD to 126.4mAHD, the time of trigger becomes progressively delayed; by 63 minutes from 3:13pm to 4:16pm. Upstream water levels that have accumulated up until the time of failure also increase by about 4.7m from 123.4mAHD to 128.5mAHD.

363. The last column in Table 10.2 lists the difference between upstream creek flood level and pit lake level at the time of levee failure. This difference starts off small at the lower trigger level and increases to a maximum of 3.7m for a trigger level of 124.5mAHD (highlighted). The difference then tends to decreases with further trigger level increase.

364. From my review of Table 10.2, I have selected two variations of the Worst Case scenario:

- Worst Case (greatest delay) – the case that produced the greatest delay until initiation of levee failure, trigger level at location Trigger D of 126.4mAHD; and
- Worst Case (greatest drop) – the case that produced the greatest Lockyer Creek to quarry pit lake drop, trigger level at location Trigger D of 124.5mAHD.

365. As I have discussed previously, all Worst Case scenarios are configured so that the levee is not overtopped until failure initiation is triggered. This, in effect, means that the levee in each Worst Case scenario is set to just higher than the level of the upstream floodwater. That is, for the 124.5m trigger scenario the levee height is at 127.1mAHD (the same height as the upstream water level). In the same way, the levee height for the 126.4mAHD trigger scenario is at 128.5mAHD (the same height as the upstream water level).

366. I have plotted the effective levee heights for both trigger 124.5mAHD and 126.4mAHD Worst Case scenarios on a survey profile of the Eastern Bund (refer to Section 8.8 for details), Figure 10.15 below.

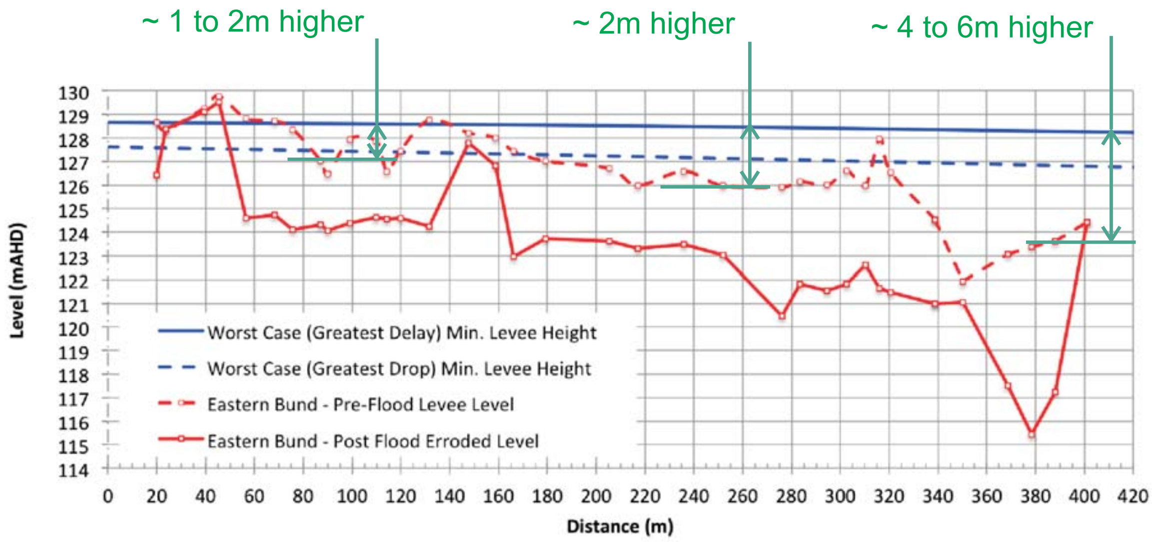


Figure 10.15 - Profile Through Eastern Bund with Worst Case Levees Overlaid

367. In Figure 10.15, the Worst Case levee crest lines are drawn in blue; a solid blue line for the Worst Case (largest delay) scenario; and a dashed blue line for the Worst Case (largest drop) scenario. I have given both lines a slight slant as the water profile in Lockyer Creek behind the levees has a gradient left to right (Lockyer Creek flow is in that direction). For reference, I have also shown in Figure 10.15 in green the approximate height that the Worst Case (greatest delay) levee sits above the original pre-flood levee profile (as of August 2010).

368. I also point out that that this suite of Worst Case scenarios all start with the pit lake full to capacity: 122mAHD. Before the flood commences the lowest point in the levees around the pit was located at the south-eastern corner of the pit. The level of the levee at this location was a little under 122mAHD, which I have adopted as the full capacity of the pit for the purposes of this analysis. This means that water will start to flow into the pit from Lockyer Creek at this location as soon as water levels in Lockyer Creek rise above this level. Reference to the pit water level column in Table 10.2 shows this occurring, with increasing pit water levels (at the time of levee trigger failure) as Trigger levels (and failure time) also increase.

369. I estimate that the creek bank level at the Quarry Road causeway is between 122mAHD and 123mAHD. Although maximum differential water level trigger level (124.5mAHD) is above this creek bank level, reference to the cross-section in Figure 10.14 shows that it is about 0.5m below the Quarry Road crown level. The significance of this road is that at this location it effectively creates a divide above which northern (left) overbank flows commence discharging eastwards towards Grantham thereby creating the Western Overbank flow.

370. I observe that the Worst Case scenario having the highest water level trigger (126.4mAHD), and the latest erosion initiation time, is a limiting case. This is because the trigger occurs at the peak of the flood and, following this time, water level declines as the flood passes. It is for this reason that I have appended *limit* to its legend entry in Figure 10.14.

371. To help me consider the relative magnitudes of flow changes associated with Quarry levee failure I examined the hydrographs of simulated flows into the Quarry through the failed section of the levee, with coincident flows in Lockyer Creek at the Quarry Road causeway, and those leaving the Quarry pit at its south-eastern corner.

372. I have plotted these in Figure 10.16:

- 124.5mAHD Trigger level - Worst Case (greatest drop); and
- 126.4mAHD Trigger level - Worst Case (greatest delay).

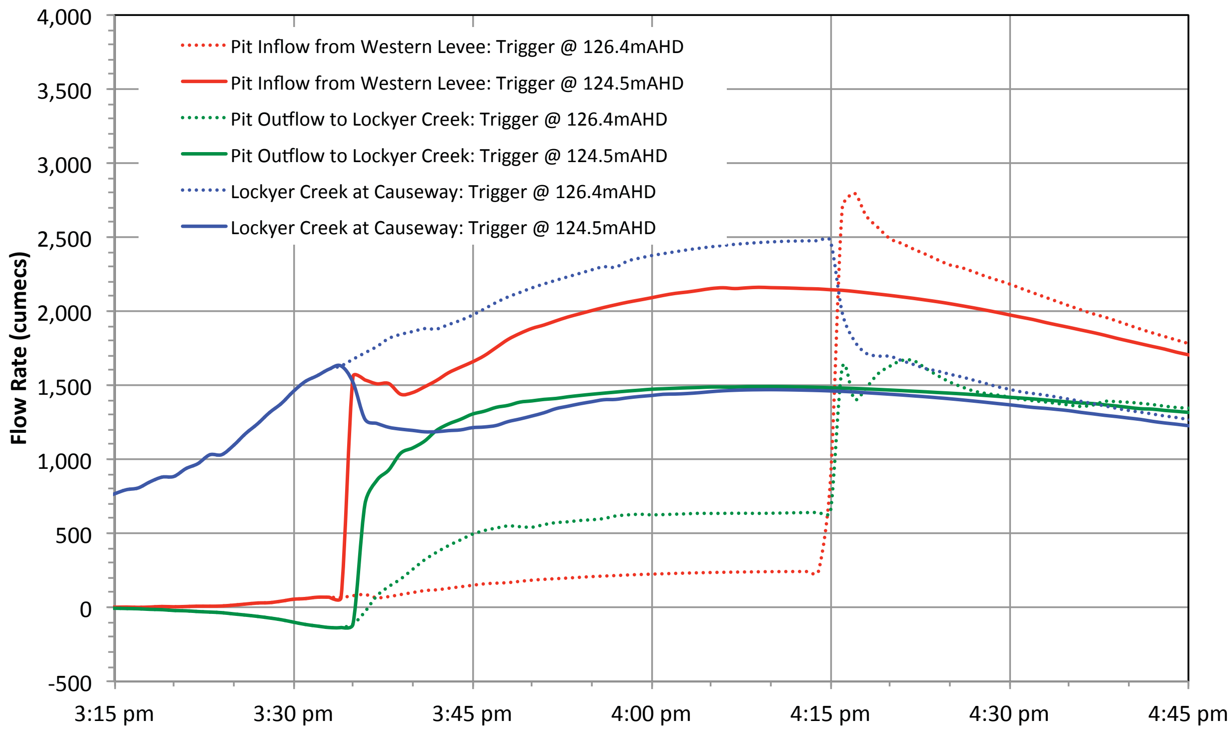


Figure 10.16 – Worst Case Quarry Breach: Selected Flow Hydrographs

373. According to Table 10.2, for a trigger at 124.5mAHD the levee should fail at 3:35pm. The trigger at 124.5mAHD is associated with the greatest drop height between upstream creek level and the pit lake level. I have set the duration of failure to 5 seconds; in other words, practically instantaneous. This failure is reflected in the flow hydrographs shown in in Figure 10.16 in a number of ways as set out in the paragraphs that follows.

374. <u>Inflow to pit (red solid line):</u>

- at 3:35pm, the inflow rate into the quarry pit rapidly increases from close to $0m^3/s$ to $1,600m^3/3$ (the small initial inflow is because some inflow is finding its way around the ends of the Western Levee prior to failure);
- the flow rate into the pit dips slightly after failure commences because of a localized rapid drawdown of flood levels immediately adjacent to the pit due to the sudden start of inflows; and
- thereafter, flow rates into the pit climb as levels in the creek continue to rise on account of the continued increasing incoming flood flow from Helidon.

375. <u>Outflow from pit (green solid line):</u>

- close examination of the plot shows that over the period preceding the triggering of levee failure (i.e. before 3:35pm) water is shown entering the pit from the south eastern corner, as indicated by the negative flow rate (negative means backwards flow); this is because water levels in Lockyer Creek adjacent to the south eastern pit are rising and causing the backflow;

- upon levee failure, outflows from the pit commence immediately, but not at the same rate as the inflows; this is because the rate of outflow from the pit is governed by water levels within the pit that need to build up over time in order to generate the outflow;
- I note that there is only very small indication of a surge in the outflow between 3:45pm and 3:50pm; the magnitude of the surge is effectively limited by the relatively large surface area of the pit lake (about 10ha) which limits the maximum rapid rise in water level at the time of levee failure to about 40mm (average inflow rate of 800m^3/s over 5 second failure time, divided into the lake surface area of 10ha); and
- I also draw attention to the pit inflow rates plotted on the graph always appearing to be higher than the outflow rates; this is because the graph does not capture a record of all inflows and outflows from the pit at other areas around the perimeter of the pit.

376. Flows in Lockyer Creek (blue solid line):

- up until the time of levee failure, Figure 10.16 shows flow rates in Lockyer Creek increasing in a manner consistent with the increase in incoming flood flows from Helidon;
- at the time of levee failure the rate of flow in Lockyer Creek is shown to decrease by around 200m^3/s over a period of about 5 minutes; and
- the reason why the drop in flow rate in Lockyer Creek flow is considerably less than that associated with the sudden rate of flow into the pit at 3:35pm is because the pit inflows are being supplied by the upstream flood storage.

377. From my review of the Worst Case (greatest delay) and Worst Case (greatest drop) flow hydrographs, I observe that:

- the general characteristics of the plotted hydrographs for the Worst Case (greatest delay) scenario are similar to those for the Worst Case (greatest drop) scenario in that the shape of the hydrographs at the times of breaching - 3:35pm for the Worst Case (greatest drop) and 4:16pm for the Worst Case (greatest delay) - are similar;
- however, there are large differences in the hydrographs associated with the delay in breach time of 63 minutes (from 3:13pm to 4:16pm). This is seen in Figure 10.16 to result in:
  - an increase in the peak rate of inflow into the pit from about 1,600m^3/s to 2,500m^3/s;
  - a decrease in the peak rate of outflow from the pit (and back into Lockyer Creek) from about 1,500m^3/s to 1,000m^3/s;
  - an increase in the peak flow rate in Lockyer Creek at the Quarry Access Road Causeway from about 1,700m^3/s to 2,500m^3/s; and
- for the Worst Case (greatest delay) scenario, after the breach has occurred (that is, after 4:16pm), the flow rate at the Quarry Access Road Causeway reduces from the above pre-breach maximum to much the same rate of about 1,500m^3/s, over a period of about 30 minutes.

378. Based on the above, I consider that the net effect of delaying the initiation of the failure of the Western Levee is to drive more of the flood flow hydrograph around the quarry (both north and south of it) than what would have occurred had the levee already breached. Then, following the occurrence of the breach, the downstream flow hydrographs rapidly tend towards what they would have been had the levee not been there prior to the flood.

379. I have produced simulated flow hydrographs for the Worst Case (greatest delay) and Worst Case (greatest drop) for the downstream locations marked in Figure 10.4. For comparison I have also included simulated flow hydrographs for the "No Quarry" case.

Figure 10.17 – Effect of Worst Case Quarry Levee Failure on Downstream Flow

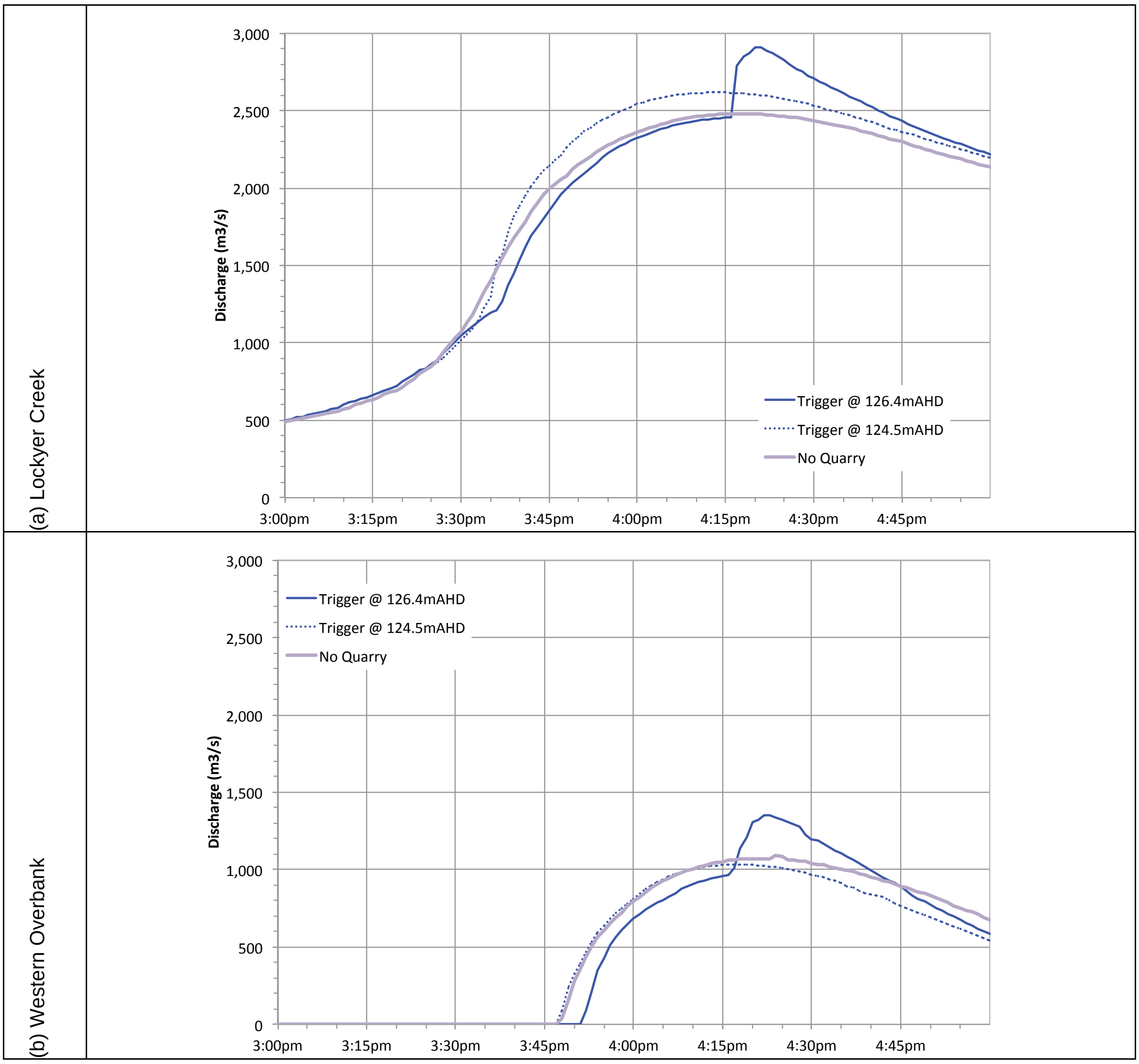
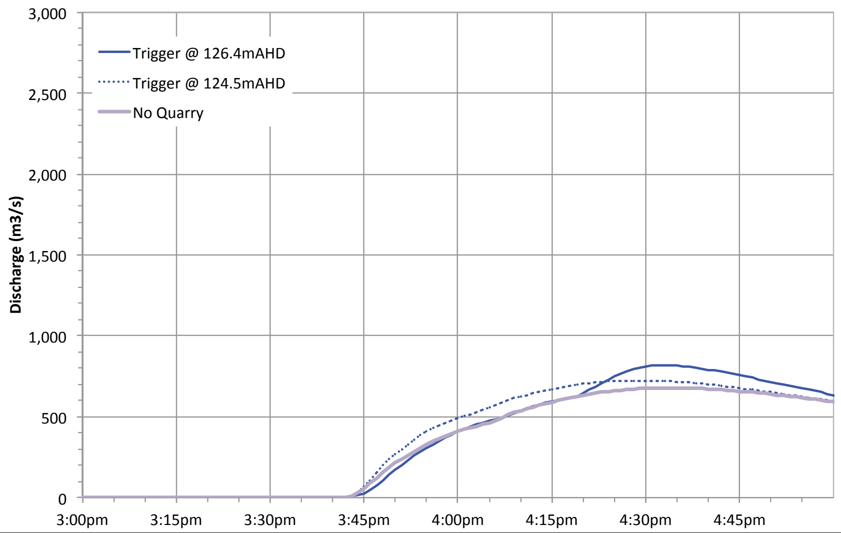


Figure 10.17 (continued) – Effect of Worst Case Quarry Levee Failure on Downstream Flow

(c) South-Western Overbank



380. In my view, the hydrographs plotted on Figure 10.17 show that the immediate effect of the levee failure in the Worst Case (greatest drop) scenario appears, in general, to be well dampened (which is indicated by the hydrograph shape which appears to be smoother and more spread-out). This contrasts to the effect of the levee failure in the Worst Case (greatest delay) scenario that appears to show much less dampening (which is indicated by the hydrograph shape which has a relatively sharp change in slope and appears less spread-out). In my view, this is because:

- for the Worst Case (greatest drop) scenario, when levee failure occurred, floodwater levels were still at below creek bank full (that is, at the Quarry Access Road causeway they were at 124.5mAHD, or about 0.5m below the crown of the adjacent road as indicated in Figure 10.14) and water levels in the pit were about 1m lower again than those at the causeway; and
- for the Worst Case (greatest delay) scenario, when levee failure occurred, floodwater levels are at around 3m higher and had already broken out of the creek at the time of levee failure (that is, at the Quarry Access Road causeway they were at 126.4mAHD, or about 0.5m above the crown of the adjacent road as indicated in Figure 10.14) and water levels in the pit were the same as those at the causeway.

381. Under these circumstances, the effects associated with a rapidly failing levee for the Worst Case (greatest drop) scenario were subject to considerably more dampening by the presence of the pit, relative to the case of the Worst Case (greatest delay) scenario for which there was effectively little or no dampening.

382. I also observe from the hydrographs plotted on Figure 10.17 that, in comparison to the No Quarry scenario, the Worst Case (greatest drop) scanario produces:

- in Lockyer Creek, a consistently greater flow with an increase of approximately 5% (140m^3/s);
- in the South-Western Overbank, peak flows are also increased by a similar relative amount; and
- in the Western Overbank, flow rates remain much the same.

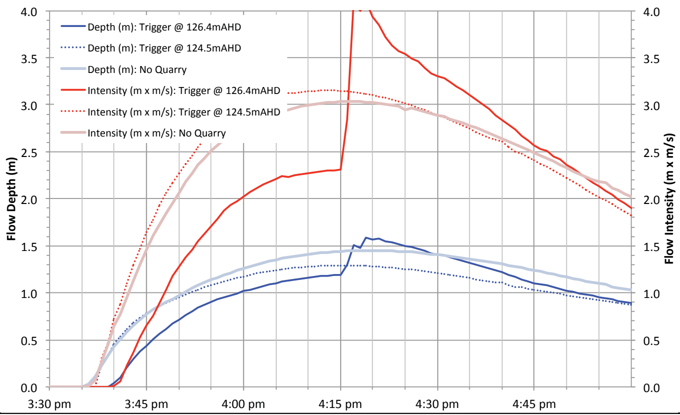
383. Further, I observe from the hydrographs plotted on Figure 10.17 that, in comparison to the No Quarry scenario, upon triggering of levee erosion (at 4:16pm, the peak of the flood), the Worst Case (greatest delay) scenario produces:

- in Lockyer Creek, a rapid rise in flow rate of about 400m^3/s (which then decreases over a period of about 30 to 45 minutes);
- in the Western Overbank, a slightly less rapid rise in flow rate of about 300m^3/s (which then decrease over a period of about 30 to 45 minutes); and
- in the South-Western Overbank, a lesser rise in flow rate of about 100 to 200m^3/s (which then decrease over a period of about 30 to 45 minutes).

384. I have also examined how the Worst Case scenarios would affect flood hazard by extracting flow depth and intensity hydrographs at the selected sampling locations marked in Figure 10.5. These are plotted in Figure 10.18 (larger copies of these graphs are replicated in Appendix C).

Figure 10.18 – Effect of Worst Case Quarry Levee Failure on Flow Depth and Intensity

(a) Near 25 Quarry Access Road



(b) 1414 Gatton-Helidon Road

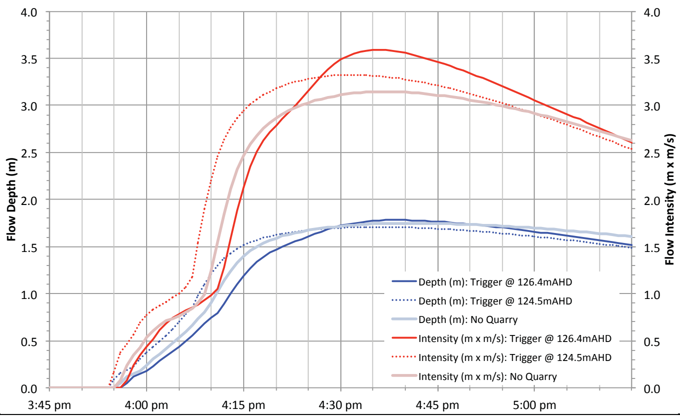
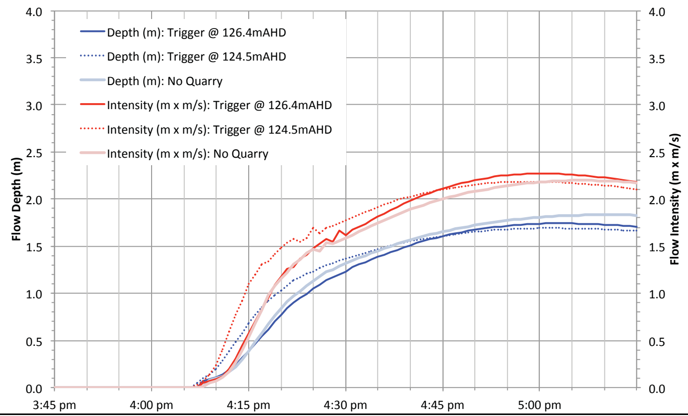
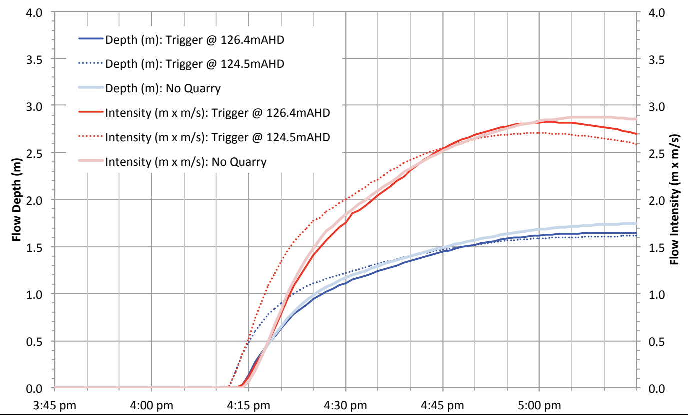


Figure 10.18 (continued) – Effect of Worst Case Quarry Levee Failure on Flow Depth and Intensity

(c) 12 Anzac Avenue



(e) 26 Anzac Avenue



385. In my view, the hydrographs plotted on Figure 10.18 show the effect of reduced dampening associated with the presence of the pit with increasing the Worst Case levee failure trigger height (as I have described above), being pronounced at near 25 Quarry Access Road, but becomes much less significant further downstream. The reason for this is that as the greater flows pass along Lockyer Creek and its overbank area it is retarded by the action of hydraulic roughness and the action of temporary storage of floodwater within the waterway. The net effect of this is to slow the flows down and spread the hydrograph out. This means that, for the case of the highest Worst Case levee failure trigger (126.4mAHD, the greatest delay scenario), the pronounced rapid increase in depth and flow intensity seen at near 25 Quarry Access Road (as shown in Figure 10.18) becomes much less apparent at the other downstream locations in Western, Central and Eastern Grantham.

386. I observe from the hydrographs plotted on Figure 10.16 that, in comparison to the No Quarry scenario, the Worst Case (greatest drop) scenario produces:

- at near 25 Quarry Access Road, a reduction in the depth of flow at the peak of the flood, and after, of about 0.2m;
- at the other locations, an increase in flow depth of up to 0.5m in the lead-up to the flood peak, but no significant change thereafter; and
- a change in flow intensity at all selected reporting locations that is generally consistent with my observations of change in depth.

387. Further, I observe from the hydrographs plotted on Figure 10.16 that, in comparison to the No Quarry scenario, the Worst Case (greatest delay) scenario produces:

- up until failure of the levee (the peak of the flood):
    - a reduction in flow depth of up to about 0.5m at near 25 Quarry Access Road, diminishing at other locations further downstream to no significant effect at 26 Anzac Avenue (Eastern Grantham); and
    - a change in flow intensity at all selected reporting locations that is generally consistent with my observations of change in depth; and
    - the rate of rise in both flow depth and intensity appears to be much the same;
- on, and after, failure of the levee at 4:16pm:
    - at near 25 Quarry Access Road, a rapid increase in flow depth of about 0.5m;
    - no significant increases in flow depth at other locations; and
    - a change in flow intensity at all selected reporting locations that is generally consistent with my observations of change in depth, noting that the peak flow intensity at the 1414 Gatton-Helidon Road is about 15% higher.

388. In my opinion, the Worst Case scenarios are unrealistic because of the underlying assumptions that I have applied. Specifically, it assumes that:

- when failure is initiated, the levee embankments all fail in unison. This is unrealistic because it assumes the Western Levee is all the same height, and constructed with the same geometry and material, which is different to Mr Starr's opinion, and the LIDAR survey of August 2010;
- levee failure to the observed post-flood surface (January 2011) happens over 5 seconds, including to the full extent of the Main Breach erosion. This is unrealistic because the amount of material removed from the Western Levee is more consistent with a 10 minute breach as I have discussed in Section 10.5 above.
- in the case of the Worst Case (greatest delay) scenario, a levee height to 126.4mAHD is much higher than contained in Mr Starr's opinion;
- an initial pit lake level to full capacity would have required an additional 2m of rainfall to have fallen at the quarry on, or within a few days prior to, the 10th January 2011 flood.

389. I recommend that the unrealistic nature of the Worst Case scenario should be kept in mind when considering simulation outcomes.

10.7 Conclusion

390. My investigations have identified uncertainty in the detail of the circumstances surrounding the timing and nature of the failure of the levee banks at the Grantham Quarry. In order to estimate the likely effect that levee failure may have had on the nature of the flood in Grantham I have therefore made my assessment on the basis of what I consider to be a reasonable range of possible conditions, including those that I judged to be the worst conditions.

391. I have examined the outcomes of the various simulation analyses in detail. This examination has included quantification of the simulated flow hydrographs both through and around the quarry and the flood flow characteristics at critical locations in Grantham.

392. To assist me with my considerations, I have overlaid the results from the Most Likely, Worst Case (greatest delay) and No Quarry scenarios onto single plots of downstream flow, and flow depth and intensity. I have then plotted from each scenario the hydrographs that I have selected to represent greatest deviation in outcome from the No Quarry scenario. The net results present an envelope of modelling outcomes and plotted in Figures 10.19 and 10.20 for the downstream flow and flow depth and intensity, respectively.

Figure 10.19 –Effect of Quarry Levee Failure on Downstream Flow

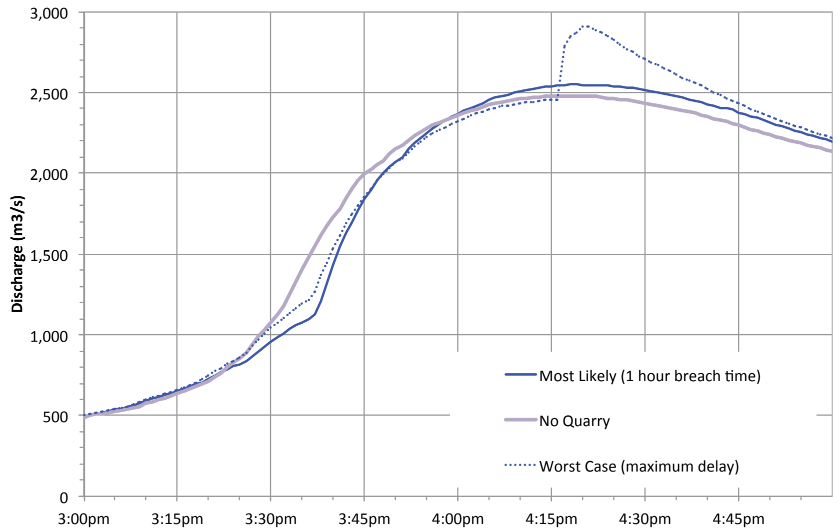
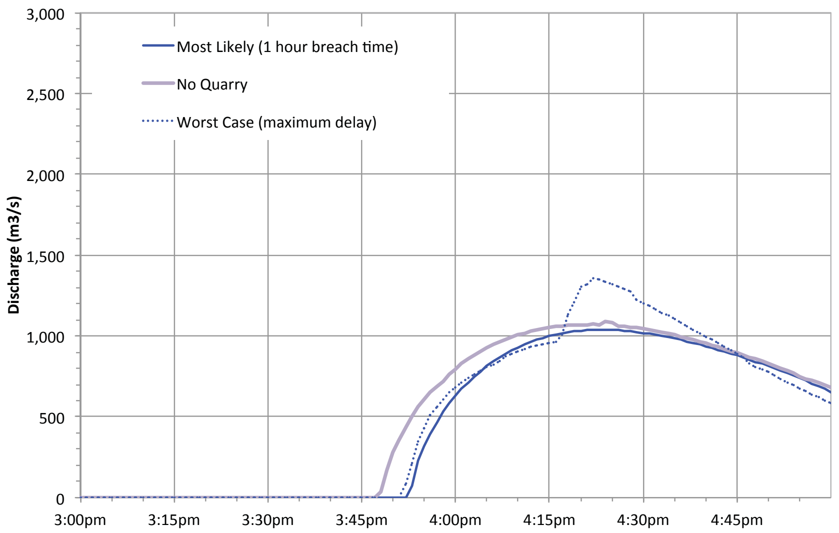
(a) Lockyer Creek	
(b) Western Overbank	

Figure 10.19 –Effect of Quarry Levee Failure on Downstream Flow

(c) South-Western Overbank

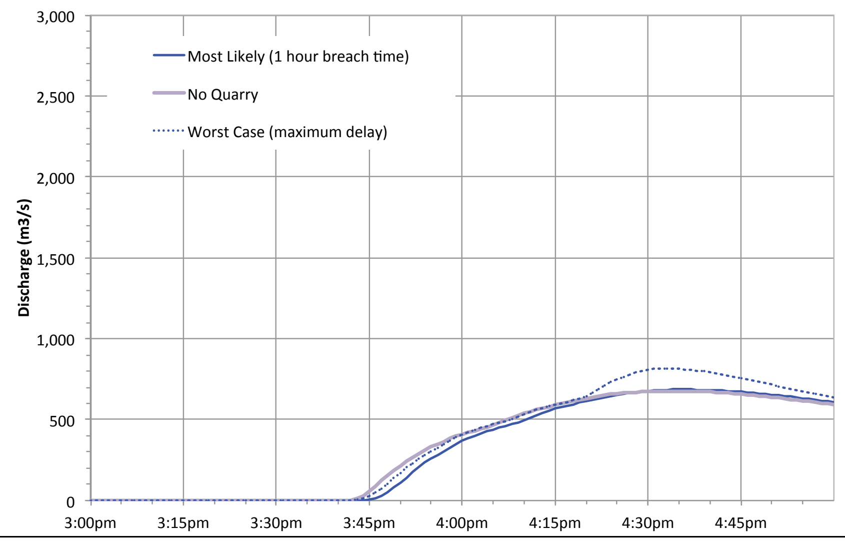
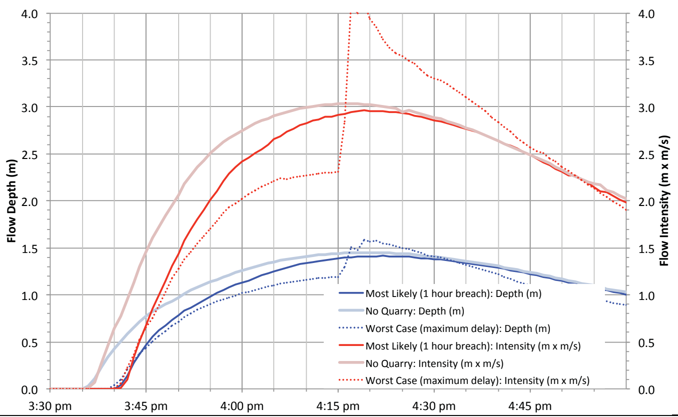


Figure 10.20 – Effect of Quarry Levee Failure on Flow Depth and Intensity

(a) Quarry Access Road



(b) 1414 Gatton-Helidon Road

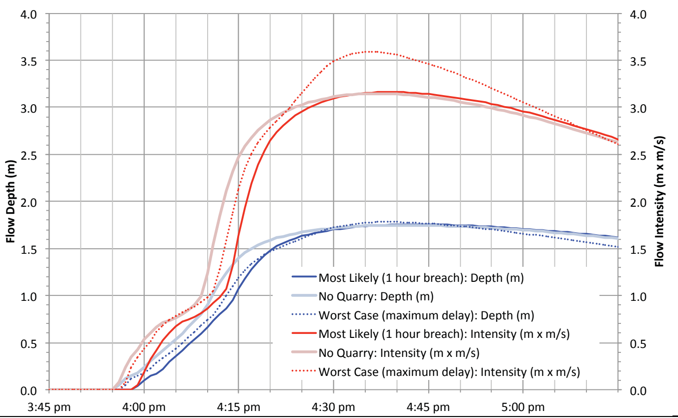
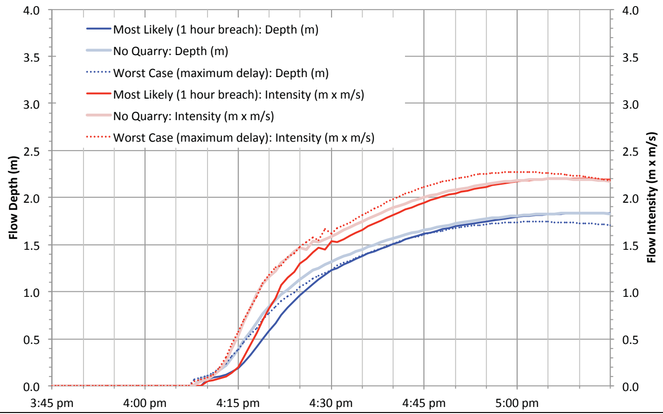
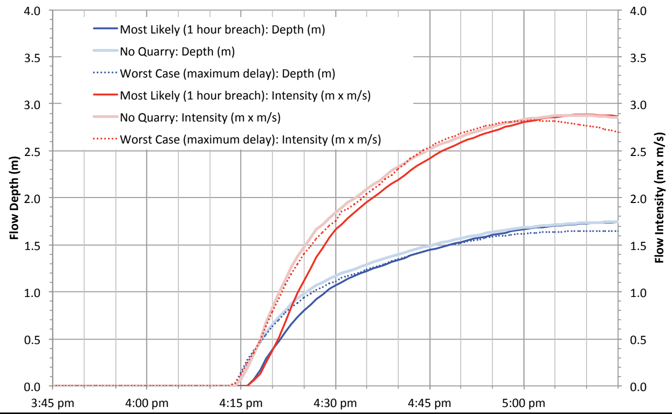


Figure 10.20 – Effect of Quarry Levee Failure on Flow Depth and Intensity

(c) 12 Anzac Avenue



(d) 26 Anzac Avenue



393. I observe from the composite set of results shown in Figures 10.19 and 10.20 that:

- levee failure would have most likely had the greatest impact in the local vicinity of the quarry with largely diminished impacts on flooding in Grantham; and
- the presence of the quarry has most likely slightly reduced the intensity of the floodwater in Grantham and delayed the time of initial inundation.

394. In considering the implications of these outcomes on flooding in Grantham I note that:

- the magnitude of the flow intensity (velocity multiplied by depth) throughout flood affected areas is likely to have been extremely high (typically above $2m^2/s$), regardless of the scenarios considered;
- a flow intensity typically above 0.5 m^2/s is of concern for vehicle safety and people; and
- peak flow depths are not significantly affected.

395. When considering the likely effect on opportunity to evacuate, other flood characteristics are of importance:

- time of occurrence of initial inundation; and
- rate of increase in flow depth and intensity.

396. I have found that time of initial inundation at Grantham is relatively insensitive to the presence of the quarry with the range being less than ±4 minutes when compared with the No Quarry case.

397. Even though this difference in time is small, I also note that its occurrence is because of changes in timing at the quarry that then propagated downstream to Grantham. That is, no one scenario introduces any “acceleration” or speedier translation of the flow hydrograph. The time difference simply relates to changes in times of occurrence at the quarry (the source of the change).

398. With exception to those locations near Quarry Access Road, my review of the rates of increase of flood characteristics has found that, at the early stages of inundation, they remain unaffected by the presence of the quarry.

399. I consider that the presence of the Quarry:

- had minimal effect on the flooding in Eastern, Central and Western Grantham in terms of flow depth, intensity, time of initial inundation and rate of increase of flow intensity; and
- accelerated the rate of rise of floodwater leading up to the occurrence of inundation and shortly thereafter at near 25 Quarry Access Road, but had minimal impact on the resulting flow depths and intensity.

400. As to the breaches of the Western Levee I have observed six primary distinct areas of erosion that have combined during the course of the flood event to form three breaches into the pit on the Western Levee:

- Main Breach – approximately 60m wide at the northern end;
- Central Breach – approximately 190m wide centrally located; and
- Southern Breach – approximately 80m wide towards the southern end.

401. These breaches total in length to 330m. I have formed the view that the Western Levee:

- allowed flood flows into the pit upon breach initiation; and
- retained and delayed the progression of the flow hydrograph up until breach initiation.

402. I hold this view because:

- the pit is located immediately downstream from the Western Levee and floodwater passes into the pit when it breaches; and
- while intact, the Western Levee restricts flows within the Lockyer Creek floodplain causing increased flood levels and retention of additional flood water upstream of the levee, thereby delaying the passage of this water further downstream.

403. The effect of the breaches on flood characteristics is demonstrated by the Most Likely and Worst Case scenarios with an effect on the timing of initial inundation. At near 25 Quarry Access Road the delay in the breaching occurrence produces a small delay of 3 to 4 minutes, and in Eastern, Western and Central Grantham it brings forward the time of inundation by up to 3 minutes.

11 Effect of the Railway Embankment on Flooding

404. Grantham is divided by a railway that runs east-west through the town. This railway is elevated above natural ground level by an embankment that extends up to a maximum of approximately 2m in height. The railway crosses Sandy Creek that also passes through the centre of the town. An underpass at the railway crossing of Sandy Creek provides the only vehicle and pedestrian connection between the north and south of Grantham.

405. I have prepared a locality map that highlights the location of the railway and Sandy Creek crossing, Figure 11.1. The natural floodplain of Sandy Creek extends to the north of the railway as indicated in the figure.

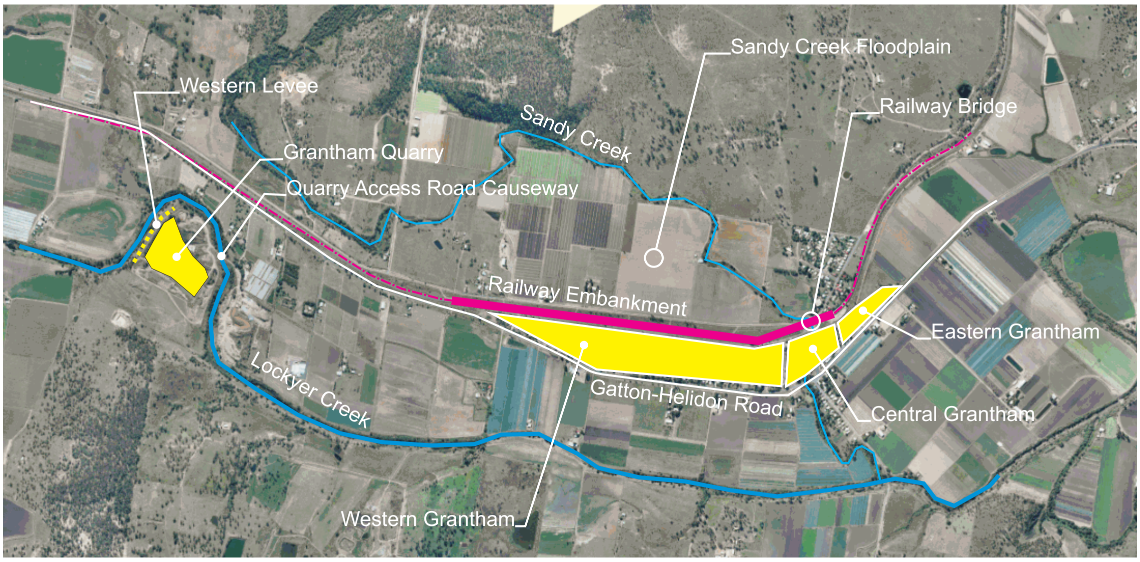


Figure 11.1 – Grantham Railway Embankment Locality

406. I have undertaken an assessment of the effect of the railway embankment on flood hazard by comparing inundation plots of maximum flow intensity (depth multiplied by velocity) for the cases of with and without the railway embankment, Figures 11.2 and 11.3 respectively. For the *No Railway* case I have removed the railway embankment by determining the natural ground levels across the width of the railway corridor. The natural ground levels were determined from topography provided by LVRC. Details of this determination are presented in Appendix B.8. These plots are colour coded and include a legend for flow intensity.

407. My review of the information presented in Figures 11.2 and 11.3 identifies a significant increase in peak flow intensity in the West Grantham area. This increase is directly attributed to the effect of the embankment confining flood flows to the south of the embankment.

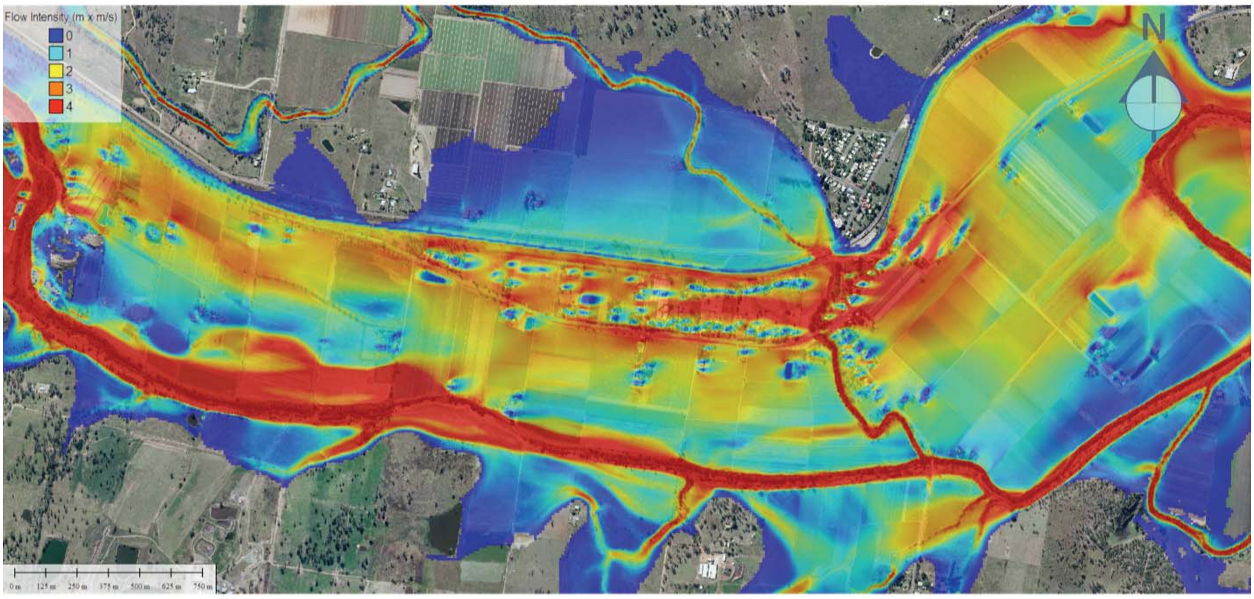


Figure 11.2 – Maximum Event Flow Intensity with Railway Embankment

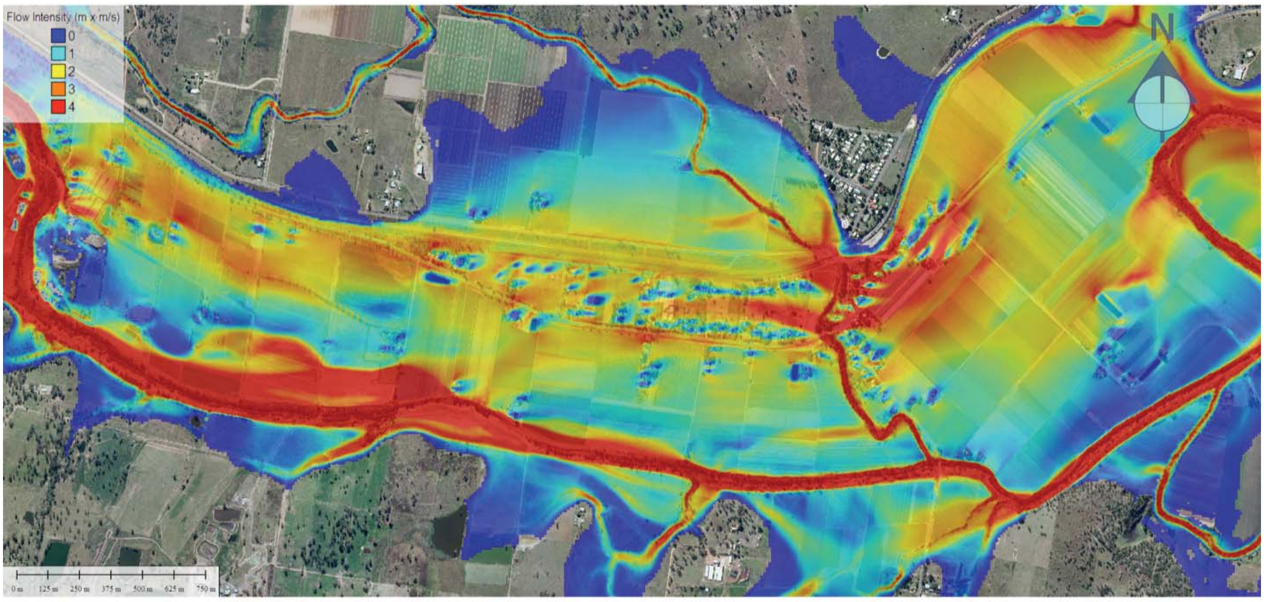


Figure 11.3 – Maximum Event Flow Intensity without Railway Embankment

408. I have then numerically subtracted the two data sets used to plot the flow intensities for the with and without railway embankment case. The resulting data set represents the increase in flow intensity on account of the presence of the railway embankment for the 10th January 2011 flood event and is presented in Figure 11.4.

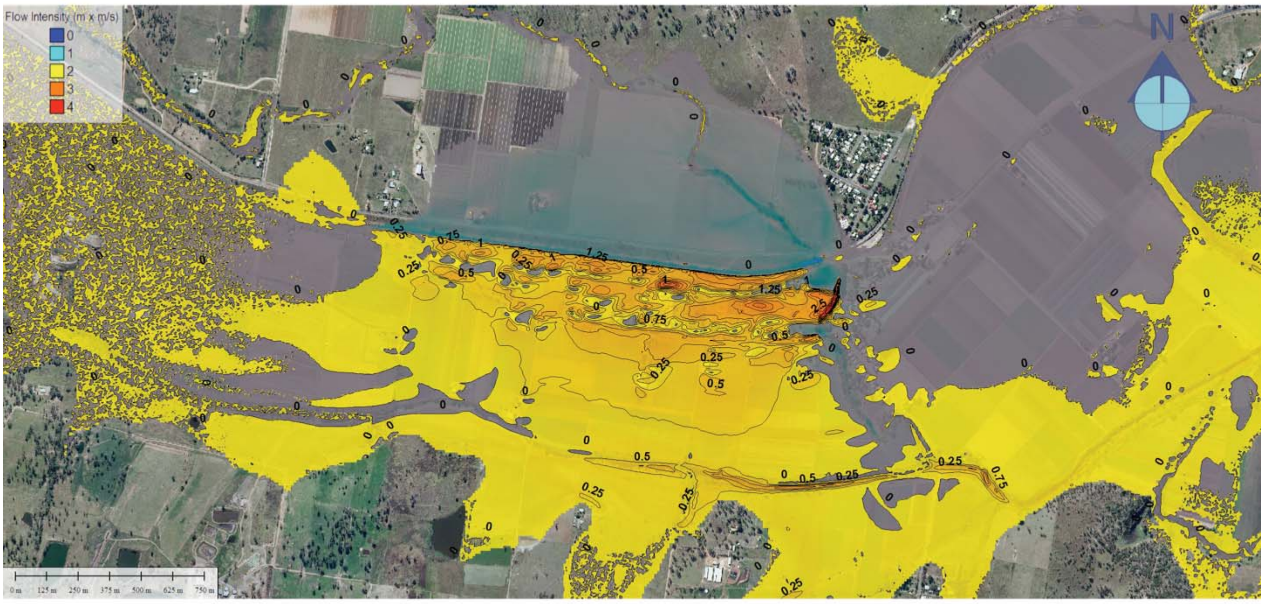


Figure 11.4 – Increase in Event Flow Intensity due to Railway Embankment

409. Based on my comparison of these cases, I consider that the effect of the railway embankment on flood characteristics would be to intercept the flow of inundating floodwaters that would otherwise have moved unrestricted into the Sandy Creek floodplain area to the north of the embankment. I have formed the view that the result of this interception was to intensify the action of flood flows within Western Grantham by:

- directing incoming flood flows from both the South-Western Overbank flow path and the Western Overbank flow path to an easterly direction, but at a greater depth and flow intensity than would have otherwise have been the case had the embankment not been there; and
- creating a concentration of flood flow to the northern side of the embankment at the location of the Sandy Creek rail bridge crossing.

410. I observe from Figure 11.4 that the railway embankment increased peak flood flow intensities through Western Grantham and Central Grantham to the west of Sandy Creek. The magnitude of the increase in flow intensity is relatively consistent throughout this area, typically by around an additional 0.5m^2/s. Closer to the western side of Sandy Creek near the rail bridge this increase becomes substantial, ranging up to an additional 2.5m^2/s.