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Detailed Two-dimensional Modelling of a Complex Bridge Arrangement – McKinlay River No. 2 Bridge, Alice Springs to Darwin Railway

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Abstract: *The Alice Springs – Darwin Railway Project involved the construction of 1420 km of new standard gauge track between Alice Springs and Darwin, including the McKinlay River No. 2 Bridge. During the 2006, 2007 and 2008 flood seasons, significant scour occurred around the McKinlay River No. 2 Bridge piers, raising concerns regarding the continuing stability of the structure. The site has complex geometry, with the river approaching the crossing at a significant angle and the remains of the original railway bridge just upstream of the new structure. Owing to the complex arrangement, a detailed 2-D hydrodynamic SOBEK model of the bridge crossing was developed to inform the design of scour protection works at the site. The model was used to analyse a number of options to reduce the potential for scour, and allowed for the scour protection works to be optimised for conditions at the site. The designed protection works were constructed in 2011, and have performed well in several subsequent flow events.*

Keywords: *bridge scour analysis, hydrodynamic modelling, SOBEK, XP-RAFTS, floodplain modelling.*

1. INTRODUCTION

The Alice Springs – Darwin Railway Project involved the construction of 1420 km of new standard gauge track between Alice Springs and Darwin and was officially opened on 17 January 2004. A number of bridges were constructed as part of this project, including the McKinlay River No. 2 Bridge (KBR 2011). Figure 1 shows the locality of the site, and Figure 2 shows the two bridges at the site at commencement of the study.

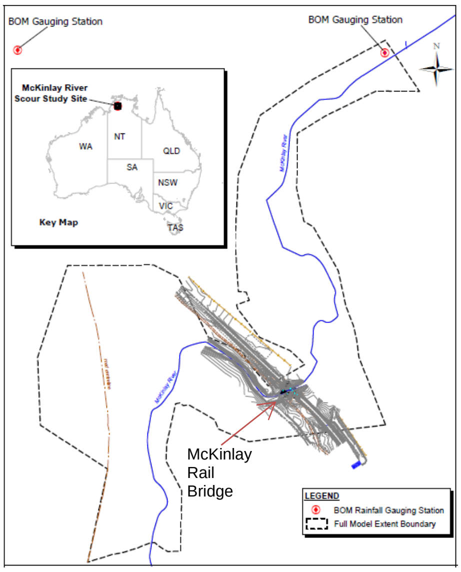

Figure 1 - The Project Study Area

During the 2006 to 2008 flood seasons, significant scour was observed around the recently constructed McKinlay River No. 2 Bridge piers (KBR 2009). Flood levels were observed close to the underside of the bridge, with severe water turbulence and whirlpool effects in areas close to the bridge. Temporary repair works were applied, but were ineffective, with about 3m of scour being identified at some piers.

The site has complicated geometry, with the river approaching the crossing at a significant angle and an old railway bridge just upstream of the new structure. Observations at the site indicate flow approaches the crossing from three directions, along the main channel of the river and also from the north and south next to the railway embankment. When these three flows interact, significant turbulence has been observed. Owing to this complexity, a detailed 2-D hydrodynamic model was developed for the site to evaluate options and assist with the scour protection design.

Figure 2 - McKinlay River – New (Concrete) and Old Bridges

2. HYDROLOGY

An XP-RAFTS model was developed of the 352 km^2 catchment area of the McKinlay River to the bridge. The modelled area is shown in Figure 3. The model was calibrated to four events: 2002, 2006, 2007 and 2008. A flood frequency analysis was undertaken based on the 53 years of data at the McKinlay River near Burrundie gauging station, and the calibration adjusted to provide an improved fit to the flood frequency results.

It was noted that the flows derived by this process were lower that the flows used in the original ADRail design, owing to the application of regional RORB parameters in the original design (AD&C-JV 2003). Given that the historical event calibration and the flood frequency both suggest lower flow rates, and the significant length of local gauged data available for this site, it is considered that the design peak discharges determined from the local data are better estimates of the 50 and 100 year events than those determined from the previously used regional approach (Hargraves *2005 & DNRM 2004).*

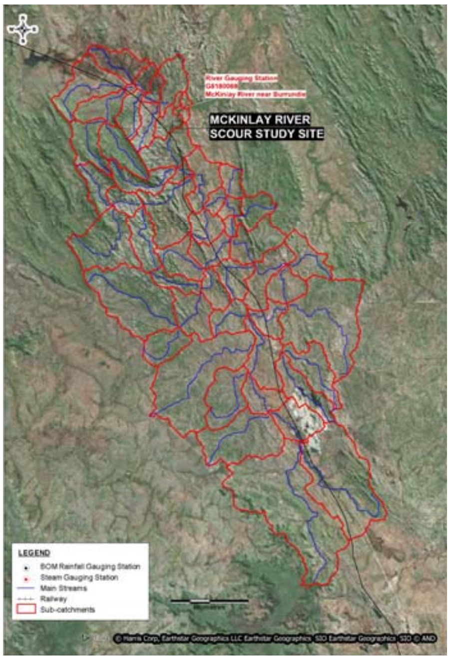

Figure 3 - XP Rafts Model

3. HYDRODYNAMIC MODEL DEVELOPMENT (SOBEK)

The hydrodynamic modelling software SOBEK by Delft was used for this project. The area modelled covers the bridges as well as approximately 2 km upstream and downstream of the bridge, with the downstream extent at gauging station G8180069. A nested grid model was used to provide an increased level of detail around the bridges. The nested grid used at bridge site was 2 m, while parent grid of 6 m used elsewhere. Water depth to grid cell size ratio exceeds four times at modelled bridge site and Sobek shallow water scheme indicate reliable answers can be obtained. Sobek model considered as most suitable due to the wall friction terms have been introduced to account for the added resistance that is caused by vertical obstacles, like piers and abutments. The wall friction coefficient is based on the average number and diameter of the obstacles per unit area and the average obstacle drag coefficient (*Deltras 2012).*

The modelled grid extents are shown in Figure 5. Upstream inflow boundaries were extracted from the XP-Rafts model, and the downstream boundary was a Q-h relationship extrapolated from the gauging station rating curve. The 2-D grid for the model was developed based on a ground survey commissioned for this project. The survey data included detailed survey around the bridge site, river cross-sections further upstream and downstream, and spot heights across flood plain areas. This data was used to build the digital terrain model (DTM) for the hydraulic model. Initially, interpolation between the available data did not provide a good representation of the river channel outside of the detailed survey area near the bridge. To solve this issue a separate HEC-RAS model was set up and used to generate interpolated river cross sections between the surveyed sections. These sections were exported to 12d Model to generate a digital elevation model for the main channel and then combined with the detailed DTM near the bridges. The remainder of the model DTM was created by interpolation from the floodplain survey points. Some adjustments were made to the resultant topography to ensure the principal topographic features of the area were well represented.

3.1. Structures

The model incorporated the old and new railway bridge structures. Modelling of the bridges was a considerable challenge for this study. In the initial set-up of the SOBEK model, both structures were modelled in 1-D, which allowed the bridge to be modelled using SOBEK's bridge routines. However, the model does not transfer momentum across the 1-D-2-D interfaces, and this caused significant instability and misrepresentation of velocities owing to the angle of the railway and river to the grid and the complex geometry around the bridges, see Figure 4.

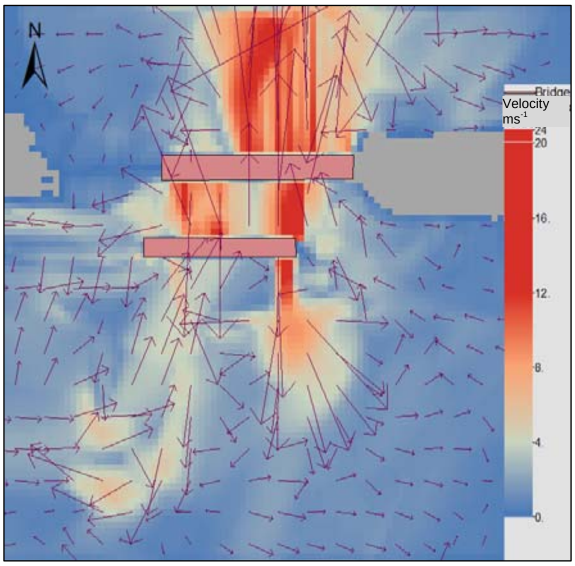

Figure 4 - Erroneous Flow Velocities at Bridge Structure –using 1-D Bridge Elements

As velocities were of primary interest to this study, it was decided to fully model the bridges in 2-D. To represent the bridge piers in the 2-D domain, two rows of piers were inserted into the Sobek model by increasing the height of cells in the approximate location of the piers. Circular bridge piers of 0.8 m &1.0 m diameter supports new and old rail bridge beams and decks respectively. The Pier rows are skewed to the flow path, which increases the chance of capturing debris around piers. Each pier row was modelled using a 4 m x 2 m grid section, which was considered to reasonably reflect the impact of the skewed pier rows and debris on flooding

The new bridge deck is out of the water in the design events of interest to this study; however, the old bridge deck is submerged by these events. The effect of the deck of the historical bridge was approximately included by increasing roughness at the old bridge location. This modelling approach was validated by comparing design flood water levels with a 1-D steady state HEC-RAS model of the two structures.

3.2. Calibration

The hydraulic model was calibrated based on the 2007 and 2008 floods, as some photography and verbal advice was available on flood levels during these events at the bridge. No actual levels were measured during events at the bridge; however, advice from the client indicated that the new bridge was not overtopped in any event, and that the highest observed water level was about 100 mm higher than as shown in Figure 5. Based on this anecdotal information, it was assessed that these two events should peak between 91.32 m and 92.25 m, with the 2008 event being higher as it was a larger event. A rainfall intensity-frequency-duration (IFD) plot was determined for these two events based on the Pine Creek rainfall station, some 5 km south of the McKinlay catchment area, (see Figure 5). At the critical duration of 18 hours for this catchment, this plot indicates that the 2007 event was smaller than the 2008 event, perhaps 1 in 2 annual exceedance probability (AEP), while the 2008 event was between a 1 in 20 and 1 in 50 AEP event.

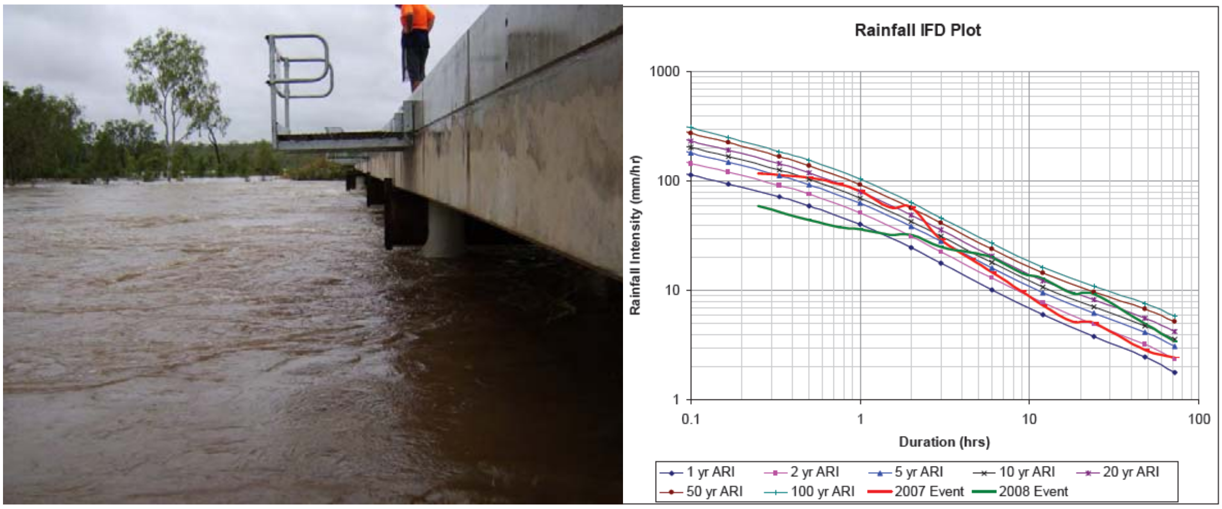

Figure 5 - Flood Level at McKinlay River Bridge (Feb 2007) & IFD Plot for Pine Creek Rainfall

Areas of similar roughness were identified from aerial photography and the site visit, and initial roughness values assigned to these areas based on experience and reference texts such as (*Chow, 1959)* and the HEC-RAS manual *(US Army Corps of Engineers 2008)* were adjusted to provide an acceptable calibration. The final roughness parameters are shown in Table 1, and the calibration results in Table 2

Table 1 - Final Manning's roughness coefficients

Description	**Manning's n**	**Description**	**Manning's n**
Dirt Road	0.030	Heavy Vegetation (around creek)	0.100
Open Land (dirt road / earth)	0.030	Rail (including embankment)	0.040
Open Land (light vegetation)	0.050	Creek	0.045
Medium Vegetation	0.070	Piers and historical bridge	0.100

Table 2 - Model Calibration

Calibration Storm	**Level at new bridge from SOBEK (mAHD)**	**Level at old bridge from SOBEK (mAHD)**	**Level at old bridge from HEC-RAS (mAHD)**	**Touching bridge deck (Y/N)**
2007	90.53	90.578	90.54	N
2008	91.64	91.755	91.68	N

The calibration for the larger 2008 event produces appropriate levels at the bridge; however, the 2007 event is significantly lower than indicated by the client's observations. Further increases in the roughness to match this event would result in unrealistically high roughness. As this study is principally interested in velocities, high roughness would also reduce velocities, which would potentially be not conservative. The poor match for the 2007 event may be due to the rainfall recorded at Pine Creek being an underestimate of the rain that actually fell in the catchment in this event.

It is acknowledged that the developed model is based on a limited data set and includes a large number of assumptions. Review of the model results indicated that the resultant velocity profiles are reasonable. It was thus considered reasonable to use the comparison of the velocity profiles produced by the model to assist in the selection of the best option to meet the objectives of this study.

4. BASE CASE RESULTS

The design events modelled in this study are the 2%- and 1% AEP events. The only changes to the SOBEK models after the final calibration for the purpose of the base case runs are changes to inflow data to reflect each design event. Design case model runs were validated against HEC-RAS model runs and original design water levels. Some anomalies in the velocity profile are present near the boundary conditions (see Figure 6), however, the results near the bridges appeared reasonable.

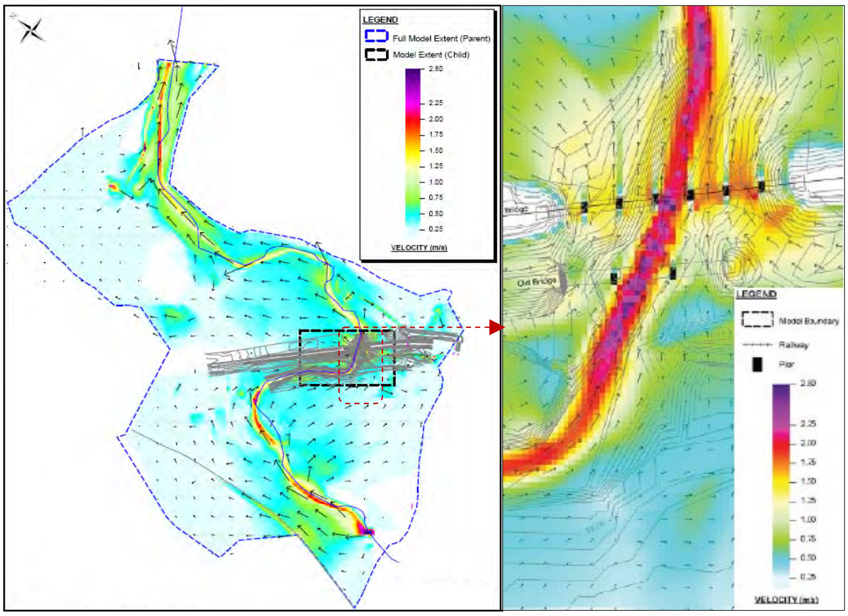

Figure 6 - Velocity Map (1% AEP, 1 in 100-year event)

The velocity patterns shown in the Figure 6 inset illustrate the locations where the high velocities occur around and between the piers, and the extent of the high velocity zones.

It was noted that the calibrated roughness values were on the high side, and thus velocities may be a little lower than in a real situation. In addition, while 2-D modelling provides a high degree of confidence in the general velocity patterns, the model may not pick up features such as localised turbulence. For this reason it was recommended that a factor of safety be applied in the design of rock protection at the site based on the modelled velocities.

5. OPTIONS ASSESSMENT

The objective of this study was to protect the bridge against scour. As velocity and flow direction is a significant contributor to scour, the focus was on reducing velocities in a controlled manner through the bridge area. The base option to address the scour issue was to apply rock protection in areas of high scour potential. However, other modifications are possible to reduce the velocity and scour at the site, and these options may be more cost effective. Four options were developed.

(i) Option 1 – Removal of left side rail embankment
This case retains the old bridge but removes the remnant rail embankment on the left side, similar to the existing arrangement on the right abutment of the old bridge. The results of this case show some moderate increases in velocity around the north side of the old bridge left bank abutment, but velocities under the new bridge are virtually unchanged. Removing this embankment thus does not provide significant savings for flows under the new bridge.

(ii) Option 2 – Removal of old bridge and abutment
This case completely removes the old bridge. The results of this case show some reductions in velocities, particularly around the former location of the right abutment for the old bridge. However, velocities slightly downstream are increased, and by the time the flow reaches the new bridge, the velocity change is minimal. In addition, the old bridge is Heritage Listed, and thus there is value in retaining it at the site.

(iii) Option 3 - High level culvert
This option assumes a culvert is provided further north, approximately where the river turns parallel to the railway embankment, to provide some relief to the flows through the main bridge at high flows. The results of this case do show a reduction of velocities at the McKinlay River Bridge, of approximately 10-20%, which would reduce the extent of the rock protection required. However, this is offset by the need for some additional protection around the entrance and exit of the new culverts. Construction of these new culverts would be costly, and may involve interruptions to rail traffic, and was thus not considered worthwhile.

(iv) Option 4 – Training walls

Option 4 involves the construction of training walls between the old bridge and the McKinlay River No. 2 Bridge, with the objective of reducing the complex flow patterns. The results of this case show a significant intensifying of flow velocities, particularly around the right abutment of the old bridge. These significant increases would increase the amount of scour protection required, and hence this option was not recommended.

The velocity patterns for each of these options are presented in Figures 7-10, along with velocity change maps compared to the base case.

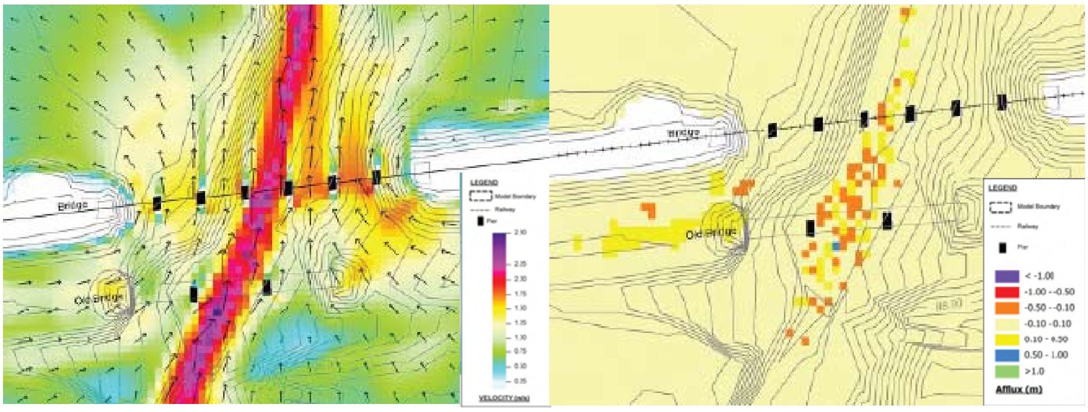

Figure 7 Option 1 (1% AEP Peak Velocity and Change in Velocity)

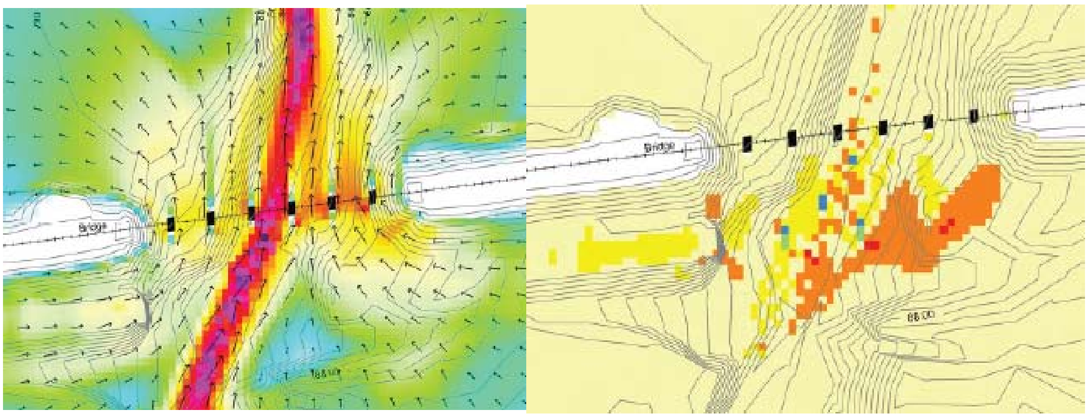

Figure 8 - Option 2 (1% AEP Peak Velocity and Change in Velocity)

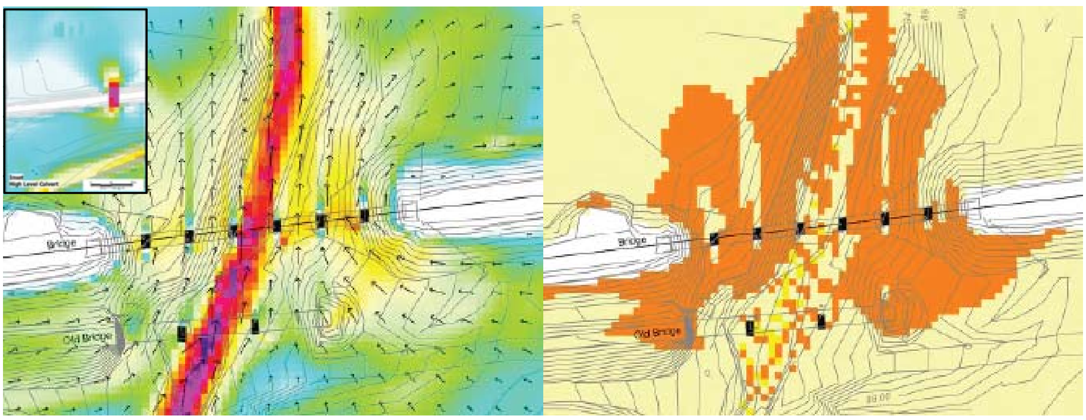

Figure 9 - Option 3 (1% AEP Peak Velocity and Change in Velocity)

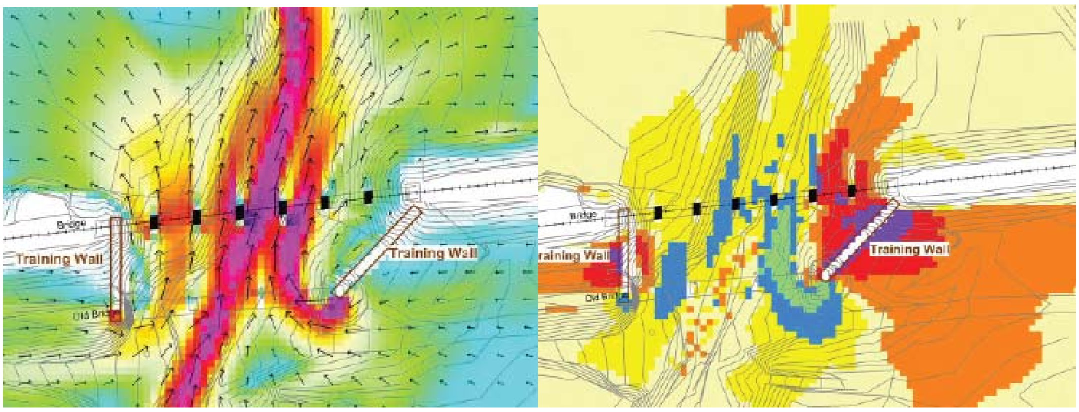

Figure 10 - Option 4 (1% AEP Peak Velocity and Change in Velocity)

6. DESIGN AND IMPLEMENTATION

The options analysis concluded that the four alternative options did not offer substantial savings over the base option of providing rock protection. A scour protection design was developed based on the results of the base case. The resultant scour protection design is shown in Figure 11, and included the following design elements:

- dumped rock protection
- Maccaferri rock gabion slope protection
- Maccaferri reno mattress
- ground rock pitching.

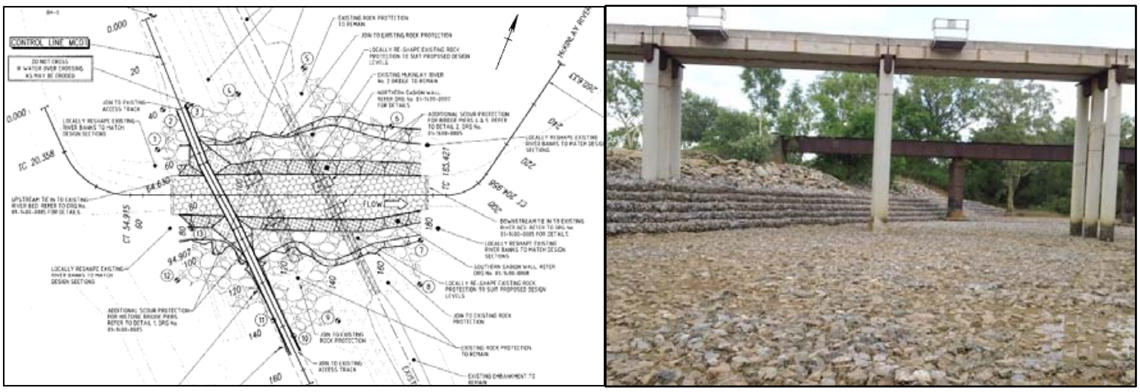

Figure 11 - Scour Protection Design and Completed Scour Protection Work

Construction on the scour protection works commenced in August 2011 and was completed in late October 2011. The approach adopted for construction was to use a check dam at the approach to the scour protection works and divert water around the construction site. A statistical assessment of possibility of flooding identified that flooding between August and October was extremely rare. No change in river level or flooding was experienced while scour protection works were completed.

Locating appropriate rock sources for the scour protection were an issue. The cheapest source of rock was located approximately 130 km from site. This partially drove the decision to adopt a rock gabion and mattress solution for the protection works given excessive transportation costs. Rock mattresses / gabions were also considered to provide a superior scour protection solution, with ability to accommodate some change in the section profile. A heavy gabion anchor on the approach was added to the design to weigh down the mattress to provide additional protection against upstream scour. Fill was replaced around the bridge piers where significant scour had occurred in line with structural assessment recommendations. Geotextile was placed for full length of scour protection construction (approximately 100 m) below mattresses. Since construction the site has performed well in several subsequent flow events. The completed works are shown in Figure 11.

7. CONCLUSION

This paper demonstrates that 2-D hydrodynamic models may be used to model bridges fully in 2-D to provide valuable information for the local design of scour protection works. The mapped velocity patterns resulting from detailed 2-D modelling are relatively easy to appreciate, illustrating the locations where the high velocities occur around and between the piers and the extent of the high velocity zones. It is highlighted that verification of such approaches against traditional 1-D approaches is strongly recommended, and that use of the information needs to consider the limitations of 2-D models in reflecting issues such as local turbulence.

The model also allowed a number of alternative options to be evaluated, including streamlining of the approach and the provision of high level culverts. As a result of the evaluation it was concluded that a rock gabion/mattress approach was the best option. Such a design was undertaken and the works constructed in 2011, and has performed well in several subsequent flow events.

The gauging station used for this study, G8180069 McKinlay River near Burrundi, was closed in 2010 because of budget cuts. This site was established in 1957, and provided a long continuous record in a region of Australia that does not have many such long-term stations. These stations provide invaluable data for a range of climate-related assessments such as this study, and the authors would like to encourage careful consideration of the long term benefits of such stations in any monitoring budget review.

8. ACKNOWLEDGMENTS

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