



Deriving Damage Indices for Concrete Girder Bridges subjected to Flood Loading

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Abstract: It is noted that the intensity and frequency of disasters have increased over the past few decades and the damage to infrastructure after a natural hazard has consequently increased. The recent flood events in Queensland, Australia had an adverse effect on the country's social and economic growth. Due to climate change impacts, it is reported that the frequency and intensity of flood events have increased noticeably in recent years. Failure of transport infrastructure after a flood event significantly affects the community, road authorities and wider stakeholders. Bridge structures are often vulnerable to flood events due to their proximity to water ways and the resultant direct impact of flood on structures. In identifying strengthening needs for vulnerable bridge structures, damage, flood intensity relationship is required.

The paper has reviewed different bridge design codes used over several years in Australia for designing the bridges and the method of design for flood loading is identified. Various failure mechanisms of bridges due to flood events have been investigated through analysis of case studies and the most common failure mechanisms of the bridges in Queensland as the result of the 2011 and 2013 flood events have been identified. A case study bridge has been modelled using the general purpose finite element software, ANSYS. The damage to bridges due to impact of floating items under different flood scenarios has been investigated. Damage curves have been generated for the case study bridge under different flood intensities.

Key Words: Bridge, Flood, ANSYS, Damage Curves

1.0 Introduction

Natural disasters such as Flooding and Bush Fire have caused significant damage to road infrastructure in Australia. The recent 2010 and 2013 flood events in Queensland impacted the country socially and economically. Frequency of flood events in Queensland appears to have increased during past decades. IBISWORLD [1] reports that the flood in March 2009 inundated 62% of the state costing \$234 million damage to infrastructure in Queensland. Theodore in Queensland was flooded three times within 12 months in 2010 and it was the first town, which had to be completely evacuated in Queensland. StateOfQueensland [2] reports that 9170 road network and 4748 rail network were damaged while 411 schools, 138 national parks and 89 bridges and culverts were destroyed during 2010-2011 floods in Queensland. Approximately 18000

residential and commercial properties were significantly affected in Brisbane and Ipswich IBISWORLD [1] during this time. The State government of Queensland and the Federal Government of Australia have incurred \$6.8 billion to rebuilding Queensland. They have paid more than \$12Million for individual, family and households while more than \$121 Million in grants for small business, primary producers and non-profit organizations. They have also paid more than \$12 Million as concessional loans to small businesses and primary produces (Rebuilding a stronger, more resilient Queensland 2012). Bridge infrastructure is vital in post disaster activities such as search and rescue operations because bridges help access to the disaster affected area Ellingwood [3]

2.0 Significance of the work

Bridge could damage in many ways when it is under an extreme flood event. Farook, Lokuge [4]. If the bridge is completely inundated during the flood, the damage to the bridge depends on the length of time it was submerged as well as the types of debris collected around or passing the bridge components. Extra care should be taken to inspect the supports of the bridges, even after the flood water recedes. Approaches of a bridge could be damaged due to debris impact, settlement or depressions. Debris against substructure and superstructure, bank erosion and damage to scour protection will damage the waterways. Bridge substructure could fail due to movement of abutments, wing walls, piers, rotation of piers and missing, damaged dislodged or poorly seating of the bearings while the superstructure could fail due to the debris on deck, rotation of deck, dipping of deck over piers or damage of girders. Pritchard [5] identified that urban debris such as cars; containers etc. and the insufficient bridge span to through that debris were the main cause for damaging bridges aftermath of 2011/2012 extreme flood events in Queensland. Figure 1 depicts some the damaged bridges from Lockyer Valley Region in Queensland.



Figure 1: Damaged Bridges in Lockyer Valley Region in Queensland

Analysis on the performance of bridges under 2011/2013 flood in Lockyer Valley Region, Queensland Farook, Lokuge [4] indicates that the bridge deck is the most commonly affected component followed by the bridge approach, pier/abutment scouring, cracks in the abutment wing walls and misalignment of abutment

headstock connections to piles. Reinforced or prestressed concrete girder bridges are a common design configuration used in Australia. During the Lockyer Valley floods in 2013, vulnerability of girder bridges was observed by significant damage to these structures.

Bridge structures have a major impact on resilience of road infrastructure and the damage to bridges could increase the vulnerability of the community served by the road infrastructure significantly. A systematic method of quantifying vulnerability of bridge structures under varying flood loading is currently a significant gap in knowledge.

Using the concrete girder bridges as case studies, the methodology to derive structural vulnerability models for bridge structures and determine vulnerable structures in the road network have been proposed.

3.0 Review of design standards

The service level of a bridge depends mainly in its load carrying capacity that is controlled by the design standards used at the time when the bridge was designed. This has resulted in bridges on the same road having different design standards and hence having different load capacities. The design



standard used to build a bridge is a good indication of the age and strength of the bridge. The design standards used in Australia can be grouped into three categories. These design standards are denoted by the descriptions of the actual design loads that were used as standard design loads such as T44, MS18 (which is a metric equivalent of ASHTO HS20) and pre-MS18 where the design standards frequently changed depending on developments in other parts of the world. All three

categories that have been described above are summarised in Table 1. Different bridges in Lockyer valley region are constructed at different times ranging from 1899 until 2010. Construction date and the possible Bridge design codes used are given in Table 2 for some of the bridges in the case study area.

Table 1: Bridge Design Standards used in (NSW) Australia. Muhunthan [6]

Design Standards - Pre-1948			
(i)	PWD	Pre-1927	Traction Engine Standard
(ii)	PWD	Pre-1927	Standard UDL
(iii)	DMR	1927	Standard UDL + Pt. Loads
(iv)	DMR	1938	Standard UDL + Pt. Loads
Design Standards - MS18			
(i)	DMR	1948	Standard Truck (MS18)
Design Standards - Post-1976			
(i)	NAASRA BDS	1976	Standard Truck
(ii)	NAASRA BDS	1976	Abnormal Vehicle Standard
(iii)	Ordinance 30C	1982	Articulated Vehicle
(iv)	AUSTROADS '92	1992	Standard T44 Truck & HLP
(v)	AUSTROADS '92	1992	HLP 320 & HLP 400 (abn.)
(vi)	AS 5100	2004	SM1600

Table 2: Bridge Design standards used for Bridges in Lockyer Valley Region. Farook, Lokuge [4]

Bridge Name	Construction Date	Codes Used
Evans Bridge	19540101	DMR'48
Weigels Crossing	19980101	NAASRA
Knopkes Crossing	19890101	NAASRA
Maincamp creek	20010101	92 AUSTROADS
Moon Bridge	19990101	92 AUSTROADS
Dotd Road Bridge	20040101	AS 5100
Main green swamp	19840101	NAASRA
Forestry Road Bridge	19660101	DMR'48
Kirsop Bridge	18991230	PWD-Pre-1927
Frankie Steinhardt's Bridge	20100701	AS 5100

3.1 AS 5100 Bridge Design Code

The AS 5100 Bridge Design Code requires that bridge over waterways be designed for flood loadings. Equations Kirkcaldie and Wood [7] are provided for determining the drag and lift forces on the superstructure for serviceability limit state and

ultimate limit state. The serviceability design flood is to be associated with a 20 year return interval. The ultimate limit state design flood is to be associated with a 2000 year return interval.

The code recommends Equation (1) and Equation (2) for calculating the drag force on the superstructure for the serviceability state (F_{ds}^*) and the ultimate limit state (F_{du}^*).

$$(F_{ds}^*) = 0.5 C_D V_S^2 A_S \quad (1)$$

$$(F_{du}^*) = 0.5 C_D V_U^2 A_S \quad (2)$$

Where V_S is the mean velocity of water flow at superstructure level for serviceability limit state (m/s); V_U is the mean velocity of water flow at superstructure level for ultimate limit state (m/s); C_D is the drag coefficient; A_S is the projected area of the superstructure (including any rails or parapets) normal to flow (m^2); and F_{ds}^* and F_{du}^* have the units of kN.

In the absence of more exact analysis, the code recommends a drag coefficient of 2.2. This is based on the research undertaken up to the time of publication of the code. The previous code, the 1976 NAASRA Bridge Design Specification, recommended a C_D of 1.4.

The code suggests that lift force may act on the superstructure when the flood stage height is significantly higher than the superstructure and the deck is inclined by superelevation. Equation (3) and Equation (4) are recommended for calculating the serviceability design lift force (F_{LS}^*) and the ultimate design lift force (F_{LU}^*) on the superstructure respectively. The equations are adapted from the equations for lift on piers.

$$(F_{LS}^*) = 0.5 C_L V_S^2 A_L \quad (3)$$

$$(F_{LU}^*) = 0.5 C_L V_U^2 A_L \quad (4)$$

Where C_L is lift coefficient depending on the angle between flow direction and the plane containing the deck (values for varying angles are quoted in code); A_L is the plan deck area (m^2).

Forces due to debris shall be calculated using Equation (5) as follows:

$$F_{deb} = 0.5 C_d V_u^2 A_{deb} \quad (5)$$

Where

A_{deb} = projected area of debris

Forces due to log impact shall be calculated as follows:

Where floating logs are possible, the ultimate and serviceability design drag forces exerted by such logs directly hitting piers or superstructure shall be calculated on the assumptions that a log with a minimum mass of 2t will be stopped in a distance of 300mm for timber piers, 150mm for hollow concrete piers, and 75mm for solid concrete piers.

Hence for the problem in question, F_{log} shall be given by the following equation (6)

$$F_{log} = mV^2/2d \quad \text{where } m= 2000\text{kg, } d= 0.075\text{m} \text{ and } V= \text{flood velocity} \quad (6)$$

4.0 Behaviour of Concrete Girder Bridges under flood loading – Numerical modelling

For the purpose of modelling the bridge, a bridge that carries a state route of Ipswich-Toowoomba road over Tenthill Creek in Gatton, Queensland, Australia has been selected. This is a simple span reinforced concrete, prestressed I-girder bridge built in 1970's. The bridge is 82.15m long and about 8.6m wide and is supported by a total of 12 pre-stressed 27.38m long beams over three spans of 27.38m. The beams are supported by two abutments and two headstocks.

General purpose finite element software, ANSYS has been used to model the bridge deck and to analyse the flood loading effect on to it. The middle span of bridge deck was analysed. All four girders were assumed simply supported and rest on the headstock of the piers. Self-weight of the bridge and the flood loads acting laterally to one of the end girder were considered in the analysis. The flood load was fed as a pressure on the face of the end girder. The bridge deck (I-girder) has been analysed using ANSYS. Figure 2 illustrates the Tenthill Creek bridge configuration. Section details of the bridge deck and the girder is given in Figure 3.



Figure 2: Tent hill Creek Bridge Configuration

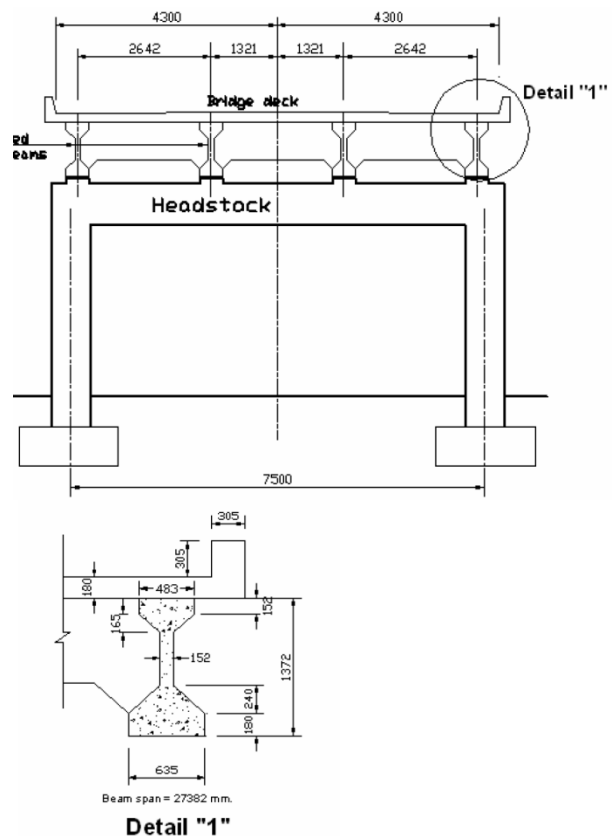


Figure 3: section details of a longitudinal prestressed I-girder beam. Nezamian and Setunge [8]

5.0 Deriving Damage Indices

Damage Indices can be derived using two methods. These indices are then used to derive damage curves for bridges under flood for various exposure conditions.

Method 1: Damage Index using structural capacity of the bridge/girder

In this method, the Damage Index (DI) is measured as the ratio between the moment capacity of the bridge girder (ϕMu) and the moment induced by flood loading on the bridge girder (M^*).

$$\text{Damage Index (DI)} = (\phi Mu)/M^* \quad (6)$$

This method requires analysis of bridge structure under the following different exposure conditions

- Bridge Elevation
- Flood Velocity
- Flood Water Level

Method 2: Damage Index using cost estimates of bridge under flood

In this method, Nishijima, Faber [9] define the Damage Index as the ratio between the repair cost and the replacement cost of the bridge under flood. Replacement cost is calculated based on the assumption that the bridge is completely damaged.

$$\text{Damage Index(DI)} = \frac{\text{Repair Cost}}{\text{Replacement Cost}} \quad (7)$$

Latter method has not been discussed in this paper.

5.1 Calculation of the existing capacity of the girder

In accordance with the Australian codes of practice for structural design, the capacity analysis methods contained in this section are based on ultimate limit-state philosophy. This ensures that a member will not become unfit for its intended use. The capacity analysis results would be compared with structural analysis results to identify the deficiencies. This approach sets acceptable levels of safety against the occurrence of all possible failure situations. The nominal strength of a member is assessed based on the possible failure modes and subsequent strains and stresses in each material.

A typical beam section of the headstock is shown in Figure 3. The positive and negative flexural and shear capacities of the section were calculated in

accordance with Australian standards (AS3600, 1988). The nominal steel rebars areas; nominal steel yield strength of 400 MPa for longitudinal reinforcement and 240 MPa for shear reinforcement and nominal concrete compressive strength of 20 MPa were used in the section capacity analysis. The degradation due to corrosion of the steel and creep and shrinkage of the concrete were ignored. Using an excel sheet, the existing moment capacity of the concrete girder section was found to be 600kNm.

ANSYS model was run for different flood velocities ranging from 0.5m/s to 5.0m/s in steps of 0.50m/s increment. Figure 4 depicts the bridge deck model used in the analysis.

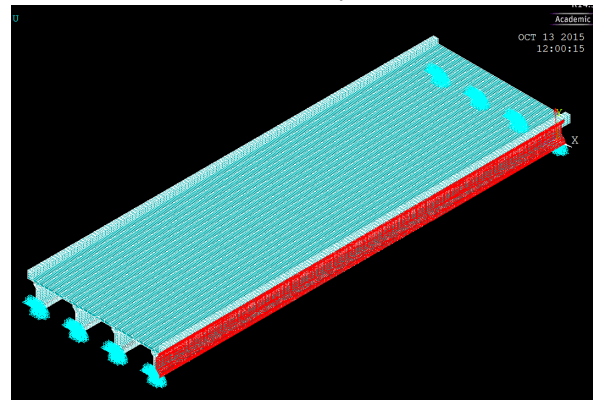


Figure 4: ANSYS bridge Deck Model

Horizontal support reaction of the end girder was obtained each time from the ANSYS Postprocessor. Using these reaction values flood induced bending moment of the end girder was calculated with the help of an excel sheet as shown in table 3

Table 3: Calculation of Flood induced bending moment (M^*)

Velocity (m/s)	Pressure (kNm ²)	Reaction (kN)	W(kN/m)	M*	$\phi Mu/M^*$
0.50	0.275	1.276	0.0932	8.544	56.18
1.0	1.1	7.223	0.527	48.355	9.93
1.5	2.475	17.133	1.252	114.71	4.18
2.0	4.4	31.001	2.265	207.61	2.31
2.5	6.875	48.848	3.568	327.04	1.47
3.0	9.9	70.652	5.161	473.02	1.01
3.5	13.474	96.42	7.043	645.53	0.74
4.0	17.6	126.15	9.215	844.57	0.57
4.5	22.275	159.85	11.676	1070.2	0.45
5.0	27.5	197.8	14.448	1324.3	0.36

$$F_d = 0.5 C_D V_s^2 A_s \text{ where } F_d = \text{kN}, V_s = \text{m}, A_s = \text{m}^2$$

$$\text{Flood Pressure} = F/A = 0.5 C_D V_s^2 \text{ where } C_D = 2.2$$

$$M_y^* \text{ (Maximum girder bending moment about minor axis)} = R_{x1} - Wx^2/2$$

Where x_1 = distance between the resultant support reaction and the half span of the girder; x = the length of the girder (=27.38m in this case)

μ = 600kNm (Existing capacity of the girder as calculated from the section analysis of the reinforced concrete girder)

ϕ = 0.8 (Safety factor for the moment capacity as per AS 5100)

Structural Vulnerability curve for this girder bridge is drawn as shown in figure 5.

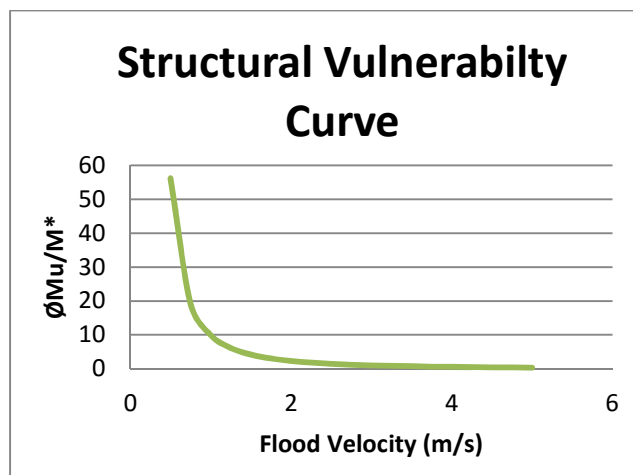


Figure 5: Structural vulnerability Curve for Tent hill Creek Concrete Girder Bridge

6.0 Conclusion

Reinforced or prestressed concrete girder bridges are a common design configuration used in Australia. During the Lockyer Valley floods in 2013, vulnerability of girder bridges was observed by significant damage to these structures. Structural performance of Tenthill Creek Concrete Girder Bridge has been studied in this paper. For the girder not to fail under flood loading, the existing moment capacity of the girder ($\phi\mu$) must be greater than the moment induced by the flood force (M^*). In other words $\phi\mu/M^* > 1$. The

critical flood velocity to satisfy this condition could be read from the above structural vulnerability curve. For the bridge considered in this case study, the critical flood velocity is read as 2.75 m/s. Outcomes will enable identification of the vulnerable girder bridges in the road networks and will assist road authorities to make optimised hardening decisions. On the other hand, emergency management services will be able to avoid vulnerable structures in determining evacuation routes.

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References

- [1]. IBISWORLD. *Queensland floods: The economic impact Special Report*. 2011 [cited 2015 10 April]; Available from: <http://www.ibisworld.com.au/404.aspx?http://www.ibisworld.com.au:80/common/pdf/QLD%20floods%20special%20report.pdf>.
- [2]. StateOfQueensland. *Rebuilding a stronger, more resilient Queensland*. 2012 [cited 2014 10 December]; Available from: <http://www.qldreconstruction.org.au/u/lib/cms/rebuilding-resilient-qld-full.pdf>.
- [3]. Ellingwood, B.R., *Assessment and mitigation of risk from low-probability, high-consequence hazards*. Australian Journal of Structural Engineering, 2009. 9(1): p. 1.
- [4]. Farook, K., et al. *Failure mechanisms of bridge infrastructure in an extreme flood event*. in *Proceedings of the 1st International Conference on Infrastructure Failures and Consequences (ICFC 2014)*. 2014. RMIT Publishing.
- [5]. Pritchard, R., *2011 to 2012 Queensland floods and cyclone events: Lessons learnt for bridge transport infrastructure*. Australian Journal of Structural Engineering, 2013. 14(2): p. 167.
- [6]. Muhunthan, B., *A PROPOSED STRATEGY FOR UPGRADING BRIDGE*

INFRASTRUCTURE NETWORK, 2008,
GRIFFITH UNIVERSITY GOLD COAST.

- [7]. Kirkcaldie, D.K. and J.H. Wood, *Review of Australian standard AS 5100 Bridge Design with a view to adoption: volume 1*. 2008.
- [8]. Nezamian, A. and S. Setunge, *Case study of application of FRP composites in Strengthening the Reinforced Concrete Headstock of a Bridge Structure*. *Journal of Composites for Construction*, 2007. **11**(5): p. 531-544.
- [9]. Nishijima, K., M.H. Faber, and M.A. Maes. *Probabilistic assessment of extreme events subject to epistemic uncertainties*. in *ASME 2008 27th International Conference on Offshore Mechanics and Arctic Engineering*. 2008. American Society of Mechanical Engineers.