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FAILURE MECHANISMS OF BRIDGE STRUCTURES UNDER NATURAL HAZARDS

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Version	Release history	Date
1.0	Initial release of document	6/12/2018

Publisher:
Bushfire and Natural Hazards CRC

December 2018

Citation: Setunge, S. et. al. (2018) Failure mechanisms of bridge structures under natural hazards. Melbourne: Bushfire and Natural Hazards CRC.

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ABSTRACT

The report captures the major failure modes of bridges under exposure to potential natural hazards in Australia: flood, bushfire and earthquake. Attributes of bridge structures which influence failure and the typical levels of natural hazards experienced in Australia are explored. Case studies on analysis of failure of bridges have been presented which can provide input to vulnerability modelling of the bridge structures.

Analysis of case studies and literature indicated that the most common failure mechanisms of bridge structures under flood is scour, debris loading and damage to approach roads. Failure mechanisms of bridges due to bushfire are significantly affected by the construction material of bridge components. Major mechanisms of failure in reinforced concrete structures is observed to be the spalling of concrete, failure due to reduction in strength and elastic modulus of concrete and yielding of reinforcing steel. In steel bridges, when temperatures rise above 400 degrees Celsius, a rapid reduction in strength of members could lead to failure. Failure mechanisms of the bridges due to earthquake are explored using analytical methods and fragility curves have been developed using finite element modeling of a bridge structure. Deck joints have been observed to be the most vulnerable elements of girder bridges in Australia under earthquake loading. The generic methodology developed will be applied to other structural forms in future.

END USER STATEMENT

Author Name, *Department Name, Organisation or Institutions Name, VIC*

INTRODUCTION

This is the third report for the Bushfire and Natural Hazards CRC project B8, entitled 'Enhancing the Resilience of Critical Road Infrastructure: bridges, culverts and flood-ways under natural hazards'. The work presented here addresses milestone 2.4.3 'Complete analysis of failure mechanisms – flood, bush fire, earthquake', and milestone 2.4.4 'Draft Report 3 – Failure mechanisms of bridge structures', which is due on 30 June 2015. Thus, this draft report will be reviewed and refined through the input of the external stakeholders, in particular Queensland Department of Transport and Main Roads (DTMR), VicRoads, RMS (NSW) and the Lockyer Valley Regional Council (LVRC).Some relevant sections of this report include extended/updated version of the Report No. 1: "Failure of road structures under natural hazards" submitted as milestone 2.1.3.

The sevenyear overall objective of the research is the development of tools and techniques to: derive vulnerability models for three types of critical road structures (bridges, culverts and flood-ways); understand the community/infrastructure interface and derive design and maintenance regimes to optimise resilience of lifeline infrastructure affecting performance of roads before, during and after a disaster. Multi-hazards of floods, fire and earthquakes are being examined including the implications of climate change and taking into account the interface between assets and community.

Australia's variable climate has always been a factor in natural disasters that have had significant impact on an evolving road infrastructure and on the communities that rely on the roads. The following figure (fig. 1) shows the average annual cost of natural disasters by state and territory between 1967 and 2005. From these data it can be seen that during this period severe storms and cyclones inflicted the most economic damage, followed by flooding. The data are strongly influenced by three extreme events - Cyclone Tracy in NT (1974), the Newcastle earthquake in NSW (1989) and the Sydney hailstorm also NSW (1999), as well as three flood events in Queensland (South East Qld, 2001: Western Qld, 2004; and the Sunshine Coast, 2005). Climate change has increased the risk from extreme events and the update of this table that includes data for the years 2007 to 2013 - during which there were extreme climate events in Qld, Vic, SA and NSW – will be of great interest to this project.

State and territory	Flood	Severe storms	Cyclones	Earthquakes	Bushfires	Total
	Cost ($ million in 2005 Australian dollars)[a]					
NSW	172.3	217.1	0.6	145.7	23.9	559.6
VIC	40.2	23.8	0.0	0.0	36.7	100.6
QLD	124.5	46.7	99.3	0.0	0.7	271.2
SA	19.3	16.7	0.0	0.0	13.0	49.0
WA	4.7	13.0	43.3	3.1	4.6	68.7
TAS	6.9	1.2	0.0	0.0	11.5	19.5
NT	9.1	0.4	138.5	0.3	0.0	148.3
ACT	0.0	0.5	0.0	0.0	9.7	10.2
Australia	**376.9**	**325.2**[b]	**281.6**	**149.1**	**100.1**	**1232.9**
Share of total *(per cent)*[c]	30.9	26.7	23.1	12.2	8.2	100.0

a. These figures exclude the cost of death and injury.
b. Figure includes costs associated with a storm involving several eastern states ($216.7 million) which has not been allocated to any individual state data in the table.
c. Figures may not add to totals due to rounding.
Source: BITRE analysis of Emergency Management Australia database <www.ema.gov.au>.

FIGURE 1: AVERAGE ANNUAL COST OF NATURAL DISASTERS BY STATE AND TERRITORY, 1967-2005 (BITRE, 2008:44)

BRIDGE STRUCTURE

Bridges

Bridges provide link between places across natural or human made obstacles such as rivers, lakes, valleys and roads. They are part of road networks and play a pivotal role in facilitating transportation flow between the network locations. Bridges are different in both their structural type and the materials used in their construction. The choice of options depends on the use and functionality of the bridge, clearance requirements and the surrounding terrain, available materials, chosen construction techniques and aesthetics.

Bridges have two main components, namely (1) superstructure and (2) substructure (Figure 2): superstructure components of a bridge are the elements which are above the supports of the bridge and provide direct function of the bridge to its users; substructure supports the superstructure elements by transferring the load to the ground. Zhaoand Tonjas [1]categorise superstructure elements into four components of wearing surface, deck, primary and secondary members (Figure 3); the substructure components consist of abutment, piers, bearings, pedestals, stem, back-wall, wing-wall, footing piles and sheeting.

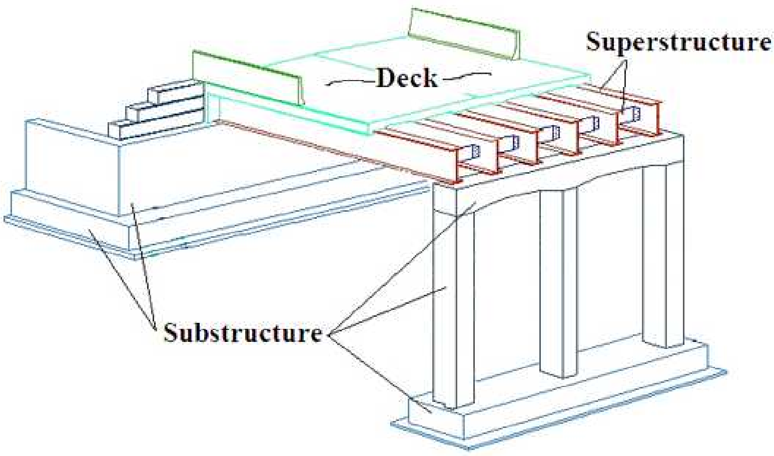


FIGURE 2: BRIDGE MAIN COMPONENTS (MDOT 2015)

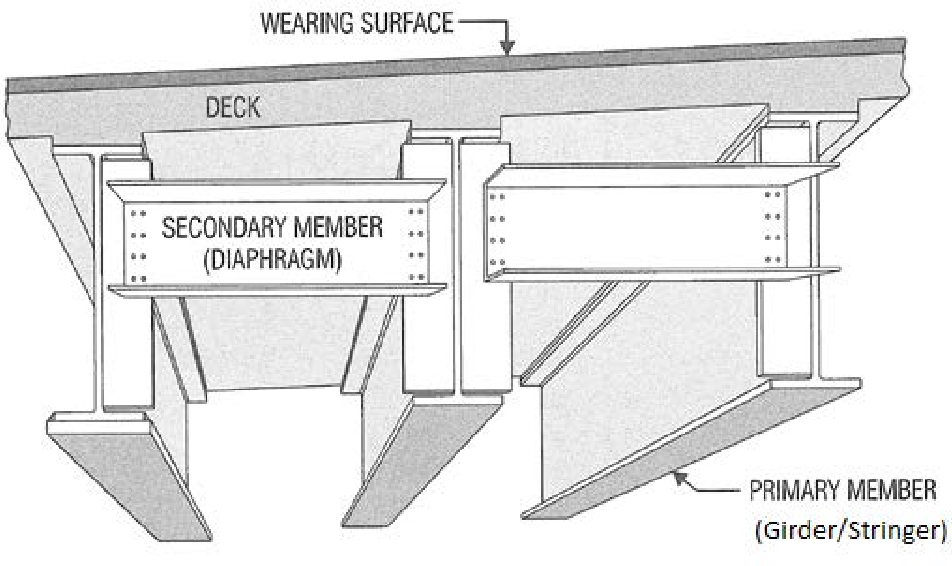


FIGURE 3: BRIDGE TYPICAL SUPERSTRUCTURE COMPONENTS [1]

ICE manual of bridge engineering[2] has explored the history of bridges and their construction materials over the years from stone, timber and masonry to steel, concrete and composite materials.

Typically, a bridge abutment is designed to resist lateral movement and overturning created by soil pressure and settlement resulting from dead and live loads. The bridge abutment and its connection to the footing must resist moments and shear forces, and the footing must provide resistance to vertical, lateral, and overturning forces. Live loads add slightly to the vertical dead loads, but they also add to the resistance to overturning and sliding. Therefore, the bridge superstructure usually controls the load ratings. However, for old bridges it is suggested that substructure is the controlling element for load rating because with long vehicles (e.g. road trains) it is possible to have 2 spans loaded where the critical load case was only one span loaded. Hence the substructure live load may be twice the design load [3]. Condition rating of concrete abutment of a bridge is mostly governed by 1. cracks or spalling due to corroded reinforcement 2. flexural cracking due to earth pressure or differential settlement of foundations 3. forward movement of abutment and 4. bearing shelf/headstock dampness. Lateral movement and rotation are caused by temperature change, friction, wind, water, and seismic loads. The bridge pier and its connection to the footing must resist moments, shear, and compressive forces. The footing must resist lateral, vertical, and rotational movements. Condition rating of a concrete pier of a bridge is mostly governed by 1. cracks due to reinforcement corrosion, 2. cracks due to moment forces, 3. cracks caused by ASR, 4. development of scour holes and 5. condition of bracing[4].

NATURAL HAZARDS

This section of the report covers the natural hazards - floods, earthquakes and bushfires. It provides an overview of each hazard and its impact on road infrastructure. Climate change is included here, although it is a risk factor for existing climate variability rather than a natural hazard in its own right. The climate change section concentrates on a general overview of the issues including dealing with uncertainty and accessing the climate science.

FLOOD

Geoscience Australia defines a flood most simply as “water where it is not wanted”, and has a more detailed description as, “a general and temporary condition of partial or complete inundation of normally dry land areas from overflow of inland or tidal waters from the unusual and rapid accumulation or runoff of surface waters from any source”. In 2011, following the widespread flooding in Queensland and Victoria the Department of Prime Minister and Cabinet of Australian Government announced that it would introduce a standard definition of ‘flood’ for certain insurance policies, reading, “the covering of normally dry land by water that has escaped or been released from the normal confines of any lake, or any river, creek, or other natural watercourse, whether or not altered or modified, or any reservoir, canal or dam.”

There has been an increase in the intensity and frequency in which flooding has occurred in Australia in the past decade. There have been a series of major floods in the Hunter Valley and Maitland (2007), Victoria and Queensland (2010 and 2011), and further more limited flooding of Eastern Australia in both 2012 and 2013.These flood events cause major disruption and damage to the built environment, particularly bridge structures.

Gourlay discusses bridge failure due to flooding in Australia in [5]:“The number of failures due to flooding is even greater in an area such as northern Australia where extreme rainfalls are very intense but infrequent and spatially highly variable. Water depths in some rivers may vary from zero to 15-20 m in a few days or even 24 h. Velocities may attain 6 m/s and flow directions may change considerably within a channel during a flood, particularly near river bends. Information concerning flood discharges and their frequency may be nonexistent. In such situations failure during floods from one cause or other is much more frequent than failure from any structural cause”.

Van Den Honert [6] analysed the flood data in Australia and illustrated the distribution across the country (Figure 1).

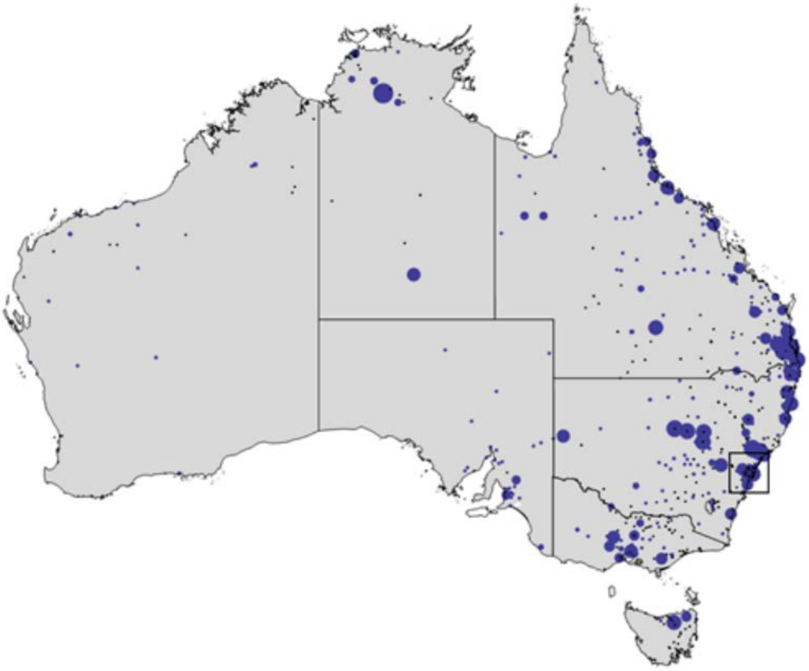


FIGURE 1: AUSTRAIAN NATURAL HAZARDS RECORDED IN PERIL AUS, 1926-2013 - BUSHFIRE (DATA SOURCE: PERILAUS DATABASE, RISK FRONTIERS)[6]

Causes of flooding have been categorised in the US in New developments and urban planning[7]. The following list uses this US information and adapts it for Australian conditions.

- Storms and cyclones
- Coastal flooding including storm surge
- Spring thaw
- Heavy rains including flash flooding
- Levees and dams failure

Road infrastructure can be affected by flooding and depending on the intensity of the flood, bridges, culverts and flood-ways can be damaged.

Flood affects the road infrastructure in several ways such as induced debris impact on the substructure or superstructure of bridges and culverts, scour and removal of the structural support, moisture ingress into the road infrastructure material and blockage of the waterway. There is also the impact of road closure during a flood and the time of repair.

The Lockyer Valley region in Queensland as one of the case studies of the project has been affected by floods and its road infrastructure has been damaged severely. Figure 2 and Figure 3 show

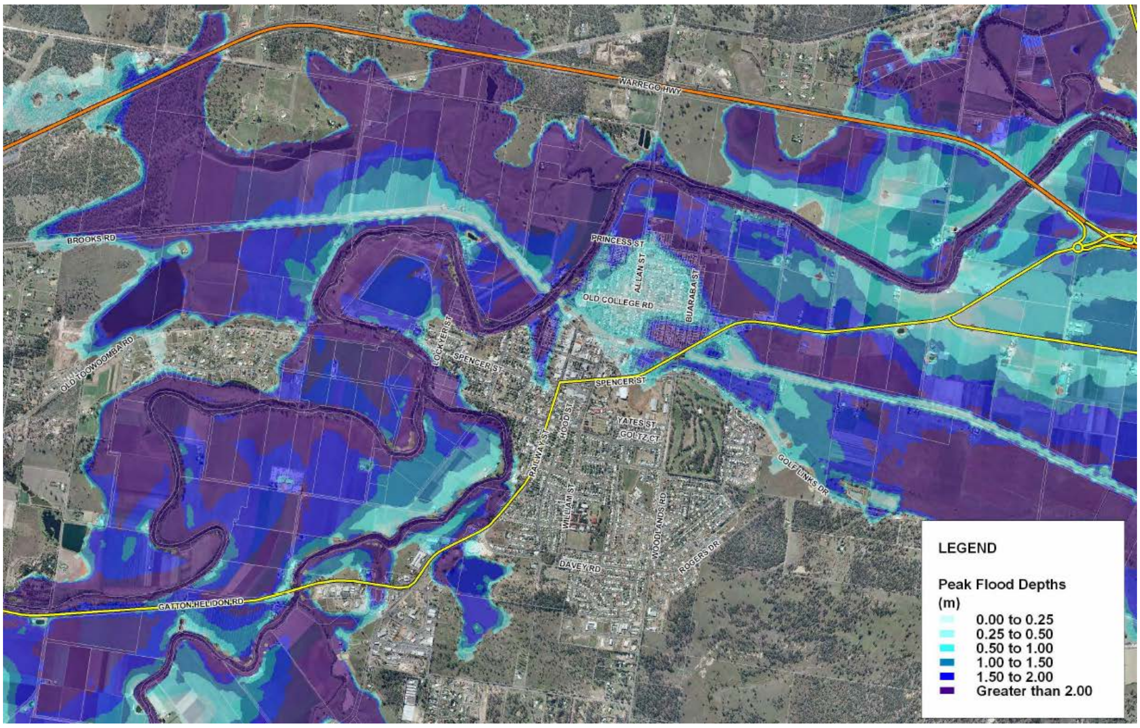


FIGURE 2: 1 IN 2000 AEP PEAK FLOOD DEPTHS - BY SKM& LOCKYER VALLEY REGIONAL COUNCIL[8]

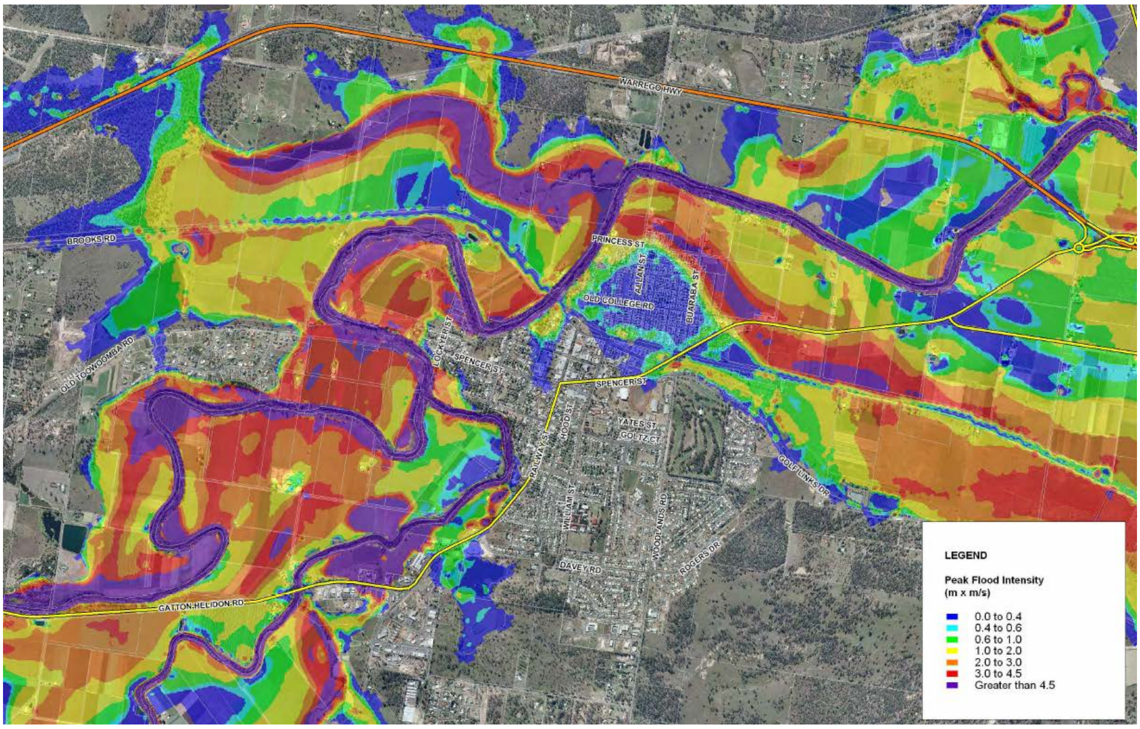


FIGURE 3: 1 IN 2000 AEP PEAK FLOOD VELOCITY - BY SKM&LOCKYER VALLEY REGIONAL COUNCIL[8]

Flooding and Forces

Australian standard for bridge design loads [9] notes that Bridges shall be designed to withstand floods up to the 2000 year average recurrence interval event without collapse or loss of structural integrity. However, for serviceability limit states it recommends that water flow forces, including those due to debris and moving objects, shall be considered for floods up to the 20 year average recurrence interval or the level of service average recurrence interval event, whichever is worse. However, code states that the design for scour should be done for more severe floods than 20 years average interval.

Flood loading on bridges - standards

92' Austroads Bridge Design Code

The 92' AUSTROADS Bridge Design Code requires that bridge over waterways be designed for flood loadings. Equations 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 the following two equations 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 $C_D {V_S}^2 A_S$

(F^*_{du}) = 0.5 $C_D {V_U}^2 A_S$

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. The following two equations 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 $C_L {V_S}^2 A_L$

(F^*_{LU}) = 0.5 $C_L {V_U}^2 A_L$

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).

AS 5100 Bridge Design Code

AS5100.2 [9]categorises the forces resulting from water flow into the flowing categories.

- Forces on pier and superstructure due to water flow
    - Drag forces

  - o Lift forces
  - o Moment on a superstructure
- Forces due to debris
- Forces due to moving objects
  - o Log impact
  - o Large item impact
- Effects due to buoyancy and lift

Following explains some of the above forces extracted from the AS 5100.2.

When a bridge crosses a river, stream or any other body of water, it shall be designed to resist the effects of water flow and wave action, as applicable. The design shall include an assessment of how the water forces may vary in an adverse manner under the influence of debris, log impact, scour and buoyancy of the structure. A few items of flood loading (categorised above) on bridge piers and bridge superstructures given in the AS 5100 standard is as follows.

<u>Flood Loading Formulae</u>

a) Drag Force on superstructures shall be calculated as follows:

F_{du} = 0.5 $C_d V_u^2 A_s$

Where

F_{du} = Ultimate design drag force

C_d = drag coefficient

A_s= wetted area of the superstructure, including any railing or parapets, projected on a plan normal to the water flow.

V_u = flood velocity.

b) Forces due to debris shall be calculated as follows:

F_{deb}= 0.5 $C_d V_u^2 A_{deb}$

Where

A_{deb} = projected area of debris

c) 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 formula.

F_{log} = mV^2/2d where m= 2000kg, d= 0.075m and V= flood velocity

AS 5100.2 [9] also states that For log and vessel impact, the relevant approach velocity is at the level of impact being considered; and for surface impact, this shall be taken as 1.4 times the average velocity.

EARTHQUAKE

Earthquake is a destructive phenomenon of natural hazard. There are two types of earthquakes namely (1) interplate earthquakes, which occur on plate boundaries and as Australia is not on the edge of the boundary it does not experience these, and (2) intraplate earthquakes, due to the movements along faults as a result of compression in the Earth's crust. Australia lies within the Indo-Australian plate and experiences these intraplate earthquakes.

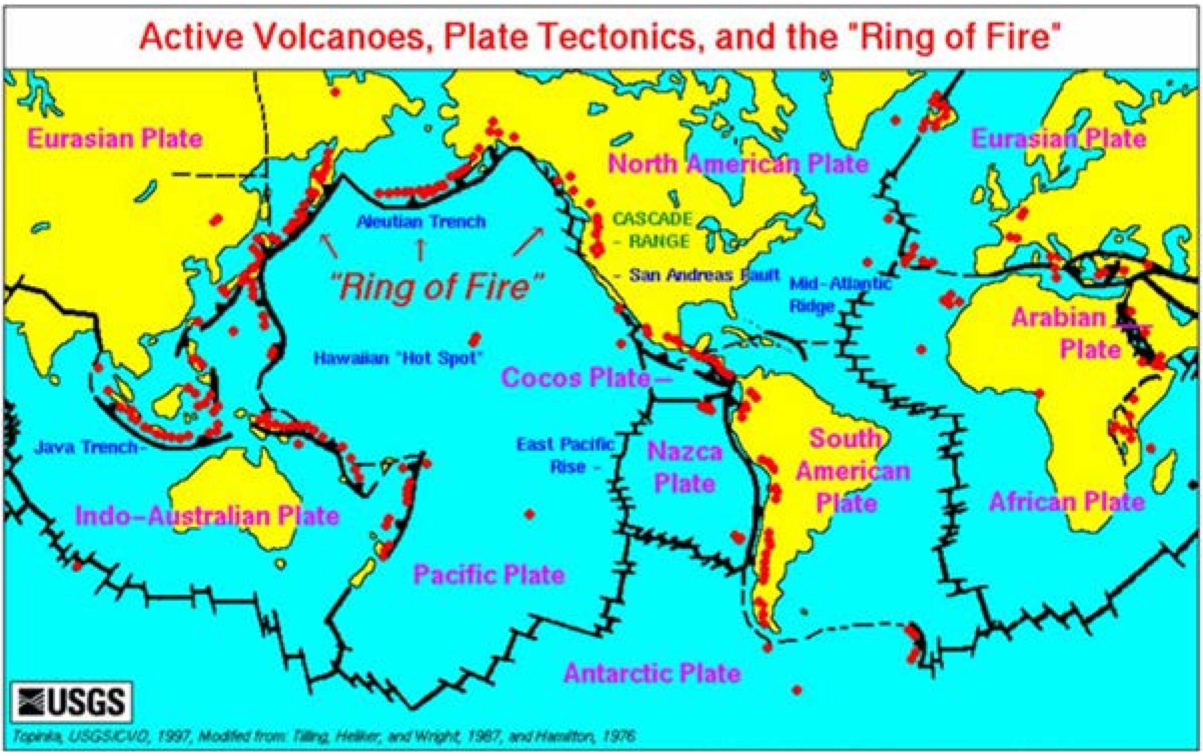


FIGURE 4: WORLD PLATE TECTONICS (HTTP://WWW.USGS.GOV)

Adelaide has the highest earthquake hazard of any Australian capital. It has experienced more medium-sized earthquakes in the past 50 years than any other capital because South Australia is being slowly squeezed in an east-west direction [10]. The Richter scale is used to measure magnitude of earthquakes and theModified Mercalli (MM) intensity scale (Primary Industries and Resources) is used to describe how people feel the earthquake. For very shallow earthquakes that are common in South Australia, with less than 10 km focal depth, the following, table presents the range [10].

TABLE 1: MODIFIED MERCALLI SCALE OF EARTHQUAKE INTENSITY

Magnitude	MM Intensity

1.2	II
2.0	III
3.0	IV
4.0	V-VI
5.0	VI-VII
6.0	VII-VIII
7.0	VIII-IX

Intraplate earthquakes are not predictable and cannot be explained from plate tectonics. Therefore the lateral loads specified on Australian structures are highly uncertain [11]. Australia is not exposed to high magnitude earthquake hazards compared to San Francisco and Wellington, however the potential for large impacts cannot be ignored.

The amount of ground motion at any given location depends on three primary factors such as distance between the site and the source location of the earthquake, known as the focus or hypocentre, total energy released from the earthquake and the nature of the soil or rock at the site. Epicentres of Australian earthquakes are shown in Figure 5.

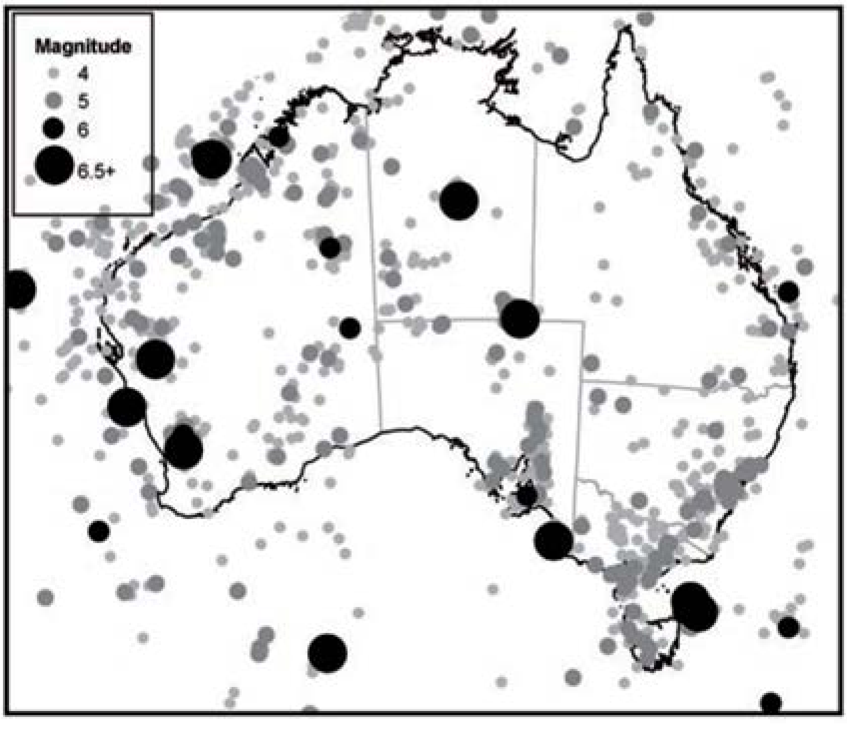


FIGURE 5: EPICENTRES OF AUSTRALIAN EARTHQUAKES 1883-2007, MAGNITUDE > 6, 1960-2007, MAGNITUDE>4 (MCCUE ET AL, 2008)

A spatially distributed earthquake source model was developed by Hall et al. [12] from spatial smoothing of historical seismicity using the earthquake catalogue described in Leonard [13]. The earthquake catalogue described by Leonard [13] based on four regions as shown in Figure 6. The regions are,

- southeasternAustralia – SEA
- south Australia - SA
- southwesternAustralia – SWA
- northwesternAustralia – NWA

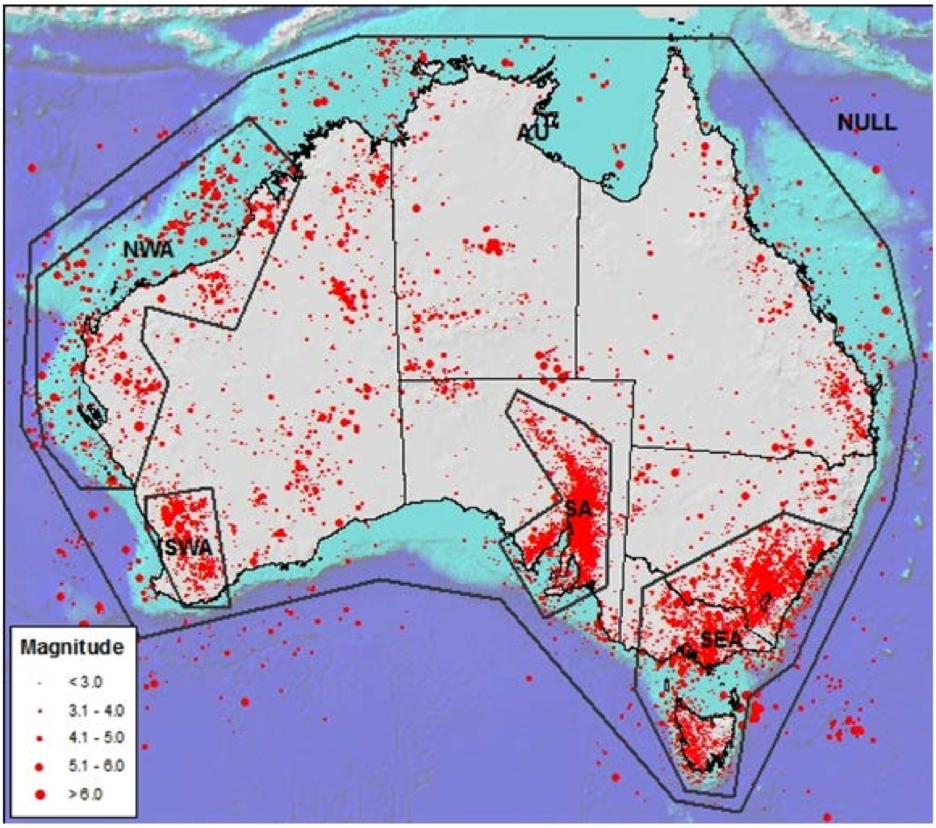


FIGURE 6: EARTHQUAKE CATALOGUE AND SEISMIC SOURCE ZONES. LEONARD (2008).

In 1989 Newcastle, NSW an earthquake with a magnitude of 5.6 was considered the largest destructive earthquake to have occurred in Australia in terms of property and life losses. In 1906, an earthquake offshore of Western Australia was recorded with magnitude of 7.2 but there was no damage due to its large epicentral distance from major population [14].

The nature of seismic activity is such that earthquakes occur at irregular time intervals and also at widely different locations. Earthquakes are rare events and structures may never experience the earthquake level assumed in the design over a lifespan. The extent and nature of the precautions that need to be taken by the community for protection against earthquakes is therefore difficult to assess, and are often controversial. It is often commonplace for the community, and even the engineering profession, to question the value of incorporating seismic provisions in the design of buildings that may never be subjected to an earthquake.

The behaviour of a structure during an earthquake depends on two basic parameters: (1) the quality of the structure and (2) the intensity of the earthquake. The structure's ability to withstand an earthquake depends on the configuration of the structural system, the design procedure, the detailing of the structural elements and careful construction. In practice, structures are not designed to completely resist earthquake loads, within the elastic range of the construction material. Rather, a dual design philosophy is incorporated in most of the seismic codes, which assumes that the structure will yield and be damaged but will not collapse during extreme ground shaking, whilst the structure will remain serviceable or operational during moderate events.

Emergency road networks also need to be considered for the management of earthquakes as interruption to their functions will greatly affect emergency and recovery activities after an earthquake. These roads can be divided into two categories including primary and secondary emergency roads. Primary emergency roads are those that make connections between national and local government offices related to disaster management and major airport or other transportation nodes. In order to set-up a primary road network, the functions of all disaster management centres have to be clearly identified and categorized. Secondary emergency roads provide links between emergency response centres (such as fire-fighting stations, police offices, hospitals and medical care centres). These primary and secondary road infrastructures including bridges, culverts, flood ways etc. need to be protected against natural hazards such as earthquake.

BUSHFIRE

Bushfires are a natural part of the Australian environment, shaping certain landscapes and resulting in fatalities recorded as far back as 110 years ago [15].Australian Capital Territory Emergency Services Agency describes bushfire as a fire that burns in grass, bush or woodland which can threaten life, property and the environment[16]. In general bushfires are uncontrolled and unwanted fire events that result from burning vegetation. The term bushfire in Australia sometimes referred as wildfire in the US or forest fire in Europe. Recent bushfire disasters in Australia include: the 1983 Ash Wednesday bushfires in South Australia and Victoria; the 2003 Canberra firestorm; the 2009 Black Saturday bushfires in Victoria; and, the recent New South Wales bushfires in 2013.

Figure 6 shows an after bushfire road infrastructure which illustrates the importance of the road network during and after a fire.

FIGURE 6: BUSHFIRE AREA - (GERAGHTY)

The effects of bushfire on road infrastructure include: immediate closure of roads and damage to the road structure and infrastructure elements. Other impacts include: damage to the surrounding area that may involve loss of stability of the surrounding area leading to landslides and erosion; danger from falling trees and the potential for future flood risk.Van Den Honert[6]analysed the bushfire data history in Australia and illustrated the distribution across the country (Figure 7).

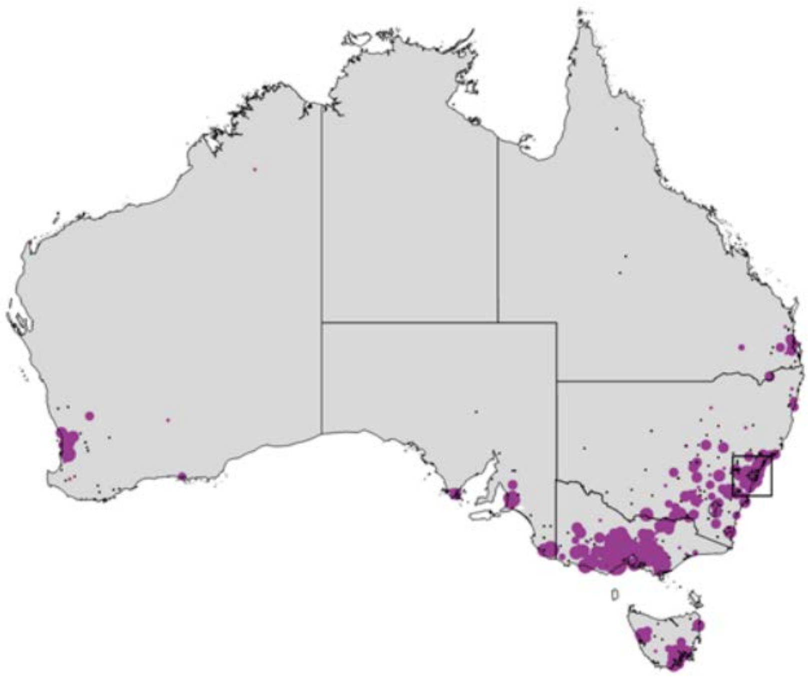


FIGURE 7: AUSTRAIAN NATURAL HAZARDS RECORDED IN PERIL AUS, 1926-2013 - BUSHFIRE (DATA SOURCE: PERILAUS DATABASE, RISK FRONTIERS)[6]

Maingi and Henry [17] have asserted that over 90% of the forest fire occurrence in their study region was caused by arsonists and human accidents. However, bushfires are also started by natural causes such as lightning strikes. Once ignited, factors such as vegetation type, wind speed, temperature, humidity and topography of the area have significant effects on the intensity and the spread of the fire. Whether they have been started intentionally, accidentally or naturally, the bushfire hazard is capable of enormous damage, threatening and casting 'a malevolent shroud over many Australian communities' [18]. Bushfires are one of the most hazardous natural disasters in Australia, ranking fourth in terms of fatalities after heatwaves, tropical cyclones and floods from the past hundred years as stated by Haynes et al. [19]. The characteristics of the continent such as low rainfall, hot and dry summers and fire proneness and dependence of eucalypt forests [20] means that bushfires will continue to remain as a major threat. Much of the country is comprised of eucalyptus forests that are fire-dependent ecosystems [21] and so fires are expected. Predicting a bushfire event and its severity with full certainty on the other hand, remains a real issue with an emphasis on ambiguity. A notable and recent example was the Black Saturday fires which on that day, were generally expected but the eventual scale and intensity of which were not wholly predicted, resulting in major consequences in terms of infrastructure and civilian lives. Additionally, climate change is increasingly becoming a key aspect of the hazard due to its effects of drier, warmer climates, enhancing the potential for fire ignition [21] and thus placing greater importance on an assured prediction of the occurrence of future bushfires.

Over the past century between 1901 and 2008, bushfires have caused a recorded total of 552 civilian fatalities, excluding the lives of fire fighters, at a rate of 5.1 every year [19]. The total number of fatalities will definitely increase with the inclusion of fatalities of fire fighters and from 2008 onwards including Black Saturday, where 173 people died, ranking among the ten worst bushfire disasters in the world in regards to mortality [22]. Withanaarachchi analysed the number of bushfire occurrences in Victorian towns from 1851 to 2014.

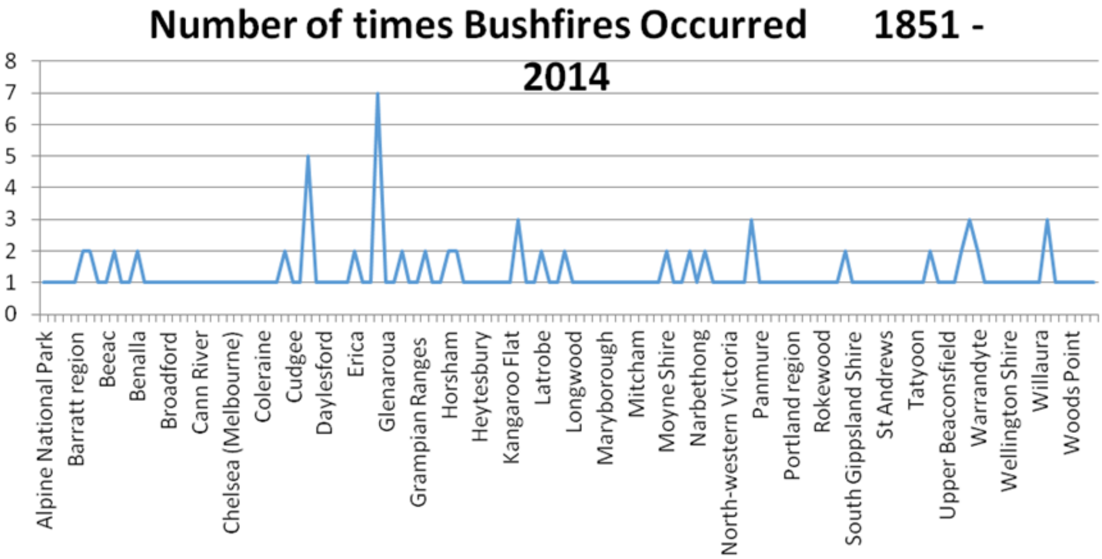


FIGURE 8: NUMBER OF BUSHFIRE OCCURRENCES IN VICTORIA (WITHANAARACHCHI J.)

Table 2 shows number of loss of lives due to bushfire in decades in Victoria.

TABLE 2: NUMBER OF LOSS OF LIVES BY BUSHFIRE IN VICTORIA (WITHANAARACHCHI J.)

Year	BushFireloss of life 1851-2011
1851-1861	15
1862-1871	0
1872-1881	0
1882-1891	0
1892-1901	12
1902-1911	0
1912-1921	0
1922-1931	31
1932-1941	71
1942-1951	61
1952-1961	32
1962-1971	105
1972-1981	6
1982-1991	52
1992-2001	8
2002-2011	179

Bushfires are the third most destructive natural hazard in terms of building losses, destroying an average of 84 buildings annually and responsible for around 20% of all losses after floods and tropical cyclones [18]. McAneney et al. [18] state that large scale fires or 'mega-fires' account for the bulk of building losses in the last 75 years with five events having recorded the most destruction. One such event, the Black Saturday bushfire, destroyed around 1834 homes, damaged thousands more and left over 7500 people without homes [22], showing the potential destruction of a mega-fire. Liu et al. [21]though, label mega-fires as only a 'recent phenomenon' accounting for 90% of burnt areas yet less than 1% of total bushfires in the US. Definition is agreed on as a fire so strong and uncontrollable that the only way to stop it is to wait until fuel runs out or for a change in weather conditions. In fact, there have been three mega-fires in Victoria already between 2002 and 2009, burning approximately 3 million hectares equal to 40% of the state's public land [20]. McAneney et al. [18]claim that losses of buildings due to bushfires are 'unlikely to alter materially in the near future' with the annual probability of losing homes from the hazard remaining relatively stable; approximately 40% annual chance of losing more than 25 homes in a single week and 20% chance of losing more than 100. They mention that this constancy is due to more urbanised living [23] and greater resources in fire fighting, education and communications. However, this does not eradicate the argument that climate change is increasing 'catastrophic wildfires globally' [21]. These dwellings may have been saved but there is no mention if the average number of mega-fires and fire fights is increasing and hence rising bushfire rates.

Below figures show the fire seasons across Australia as well as the normal/above normal bushfire potential predicted for the country (Figure 10). Figure 11 and Figure 12may illustrate some underlying reasons for estimation of higher intensity bushfires in Australia. The 2014 rainfall declines (Figure 11) show the below average rainfall areas on the east, centre-east and the west areas of the country where they almost match the above average temperatures recorded during 2014 by the Bureau of Meteorology (Figure 12).

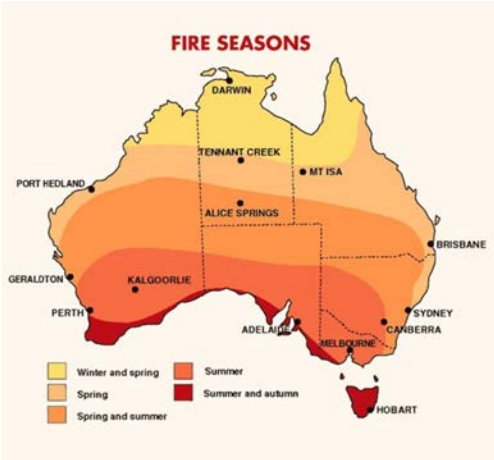


FIGURE 9: FIRE SEASONS IN AUSTRALIA (WWW.GA.GOV.AU)

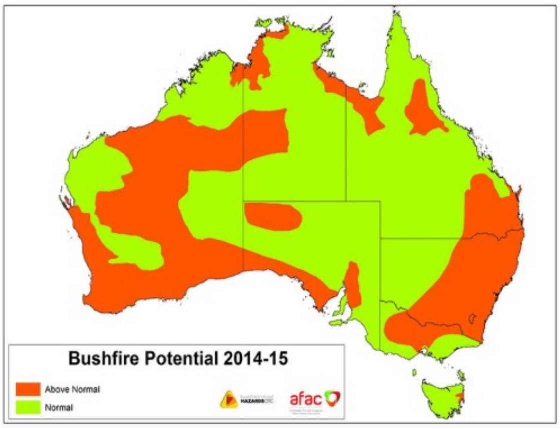


FIGURE 10: BUSHFIRE POTENTIAL (BUSHFIRE OUTLOOK WWW.BNHCRC.COM.AU)

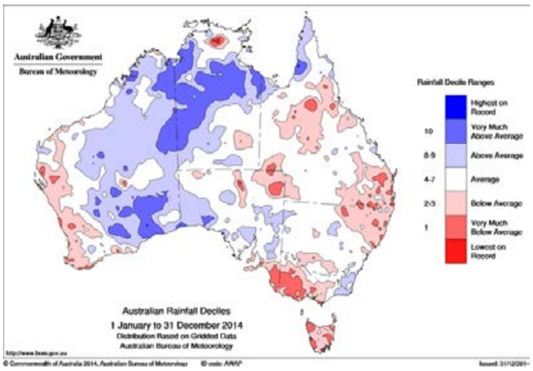


FIGURE 11: RAINFALL DECILE RANGES JAN-DEC 2014 (WWW.BOM.GOV.AU)

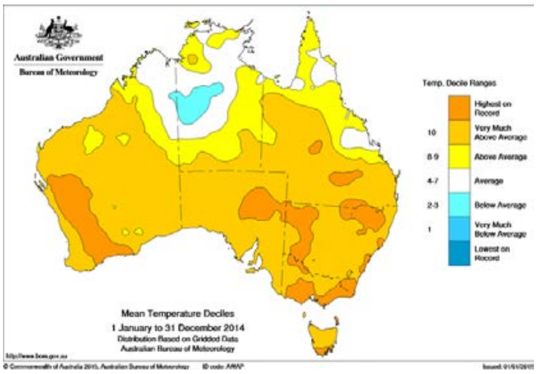


FIGURE 12: MEAN TEMPERATUREDECILES RANGES JAN-DEC 2014 (WWW.BOM.GOV.AU)

Key concepts, factors and variables

Bushfire potential and behaviour can be affected by a range of things where key determinants for bushfires include weather, climate, fuel properties and topography, as stated by Liu et al. [21]. The most common variables used are combinations of temperature, relative humidity, wind speed and drought effects [24], with the majority of these employed by fire danger systems including the system in Australia. Preston et al. [25]suggest that humidity and rainfall reduction and increasing temperatures and wind speeds are generally favourable conditions for bushfire risk[26]. Since these are quantifiable variables, they can be measured and gathered from meteorological networks to be used in quantifying bushfire potential and probability.

General temperatures, dry-bulb temperature and relative humidity are important factors correlated with bushfire potential as they influence burn rates [25]. They have an effect on the moisture content of fuels, increasing moisture content with increases in relative humidity and reduction of moisture content as temperature increases [24]. The significance of these parameters is agreed upon by Liu et al. [21] as they mention that high temperatures and dry weather are beneficial for intense fires along with strong wildfire emissions. Sharples et al. [24]assert that the instinctive concept that hotter and drier conditions, component of drought effects, correspond to lower fuel moisture contents is supported by the fuel moisture index.

Wind speeds add another dimension to fire risk, as Cheney et al. [27] mentioned that wind speed can affect the rate of spread in forest fuels. This has been backed up by an experimental fire that had a spread rate five times faster than observed on another individual fire, where they concluded that it was due to 'strong convection-induced wind' [27]. Sharples et al. [24] established fire danger rating models that incorporated wind speed as a component, mentioning that wind is the most critical meteorological factor affecting fire potential and also claiming that it is central in determining rate and direction of fire spread. Indeed, they mention that fire danger rating can be instinctively conceptualised as 'wind speed divided by fuel moisture content' or by simple examination of wind effects on vegetation.

Fuel properties and topographic attributes are determinants of bushfires as they affect the rate at which fires spread, supported by Preston et al. [25] who report that the landscape sensitivity to bushfires were influenced largely by vegetation and topography. They mention that research on the effects of topography on fire behaviour found upward slopes caused fires to burn faster and stronger than on flatter landscapes while downward slopes caused a slower fire spread. Fuel properties including fuel load are important factors in bushfires as they stimulate flames, formulating the system of prescribed burning of fuel loads to minimise their effects on bushfires [28].

Existing theories and research

Fire danger is an extensive concept that correlates to probability of bushfire ignition and propagation thus, fire danger rating systems are utilised to evaluate the potential for bushfire occurrence, spread of fire and difficulty of fire control [24]. Various factors affect fire danger where rating systems consist of a number of fire danger indices that quantify the risk of fire occurrence by employing factor combinations, varying in different countries and regions due to diverse environments. The most commonly used fire danger system used in eastern Australia is the McArthur Forest Fire Danger Index (FFDI) as outlined by Sharples et al. [24], where the index is maximised at 100 and accompanied with the McArthur forest fire danger meter that classify the FFDI into fire danger conditions such as low, medium, high, very high or extreme. Low corresponds to an FFDI range of 0-5, moderate for 5-12, high for 12-25, very high for 25-50 and extreme for 50 and over, with suppression difficulties ranging from easily suppressed fires for low to 'virtually impossible' for extreme as reported in[26]. The FFDI expression integrates factors of dry-bulb temperature in Celsius, relative humidity as a percentage, wind speed in kilometres per hour at 10 metres above ground level and the drought factor [24] ranging from 1 to 10. Other fire danger indices include the Grassland Fire Danger Index (GFDI), Forest Fire Weather Index (FWI) used in Canada and the Keetch-Byram Drought Index (KBDI) used in the US [21], each differing due to the use of factor combinations and thus quantified calculation. The various number of fire danger indices makes it rather difficult to employ one system for fire classification or potential.

Penman and York mentions in[28] that predicted changes to global climates are anticipated to have an impact on various natural systems, with natural fire systems being one of these. Although the relationship between weather, climate and bushfire potential is complex, projected effects of climate change on temperature and wind speed increases and average annual rainfall and relative humidity decreases, will lead to greater extreme fire weather conditions, escalating the frequency and severity of bushfires in Australia [25]. The consensus in general conclude that climate change will play a major role in bushfire potential with research showing that this potential increases significantly in Australia due to warming [21]. Alexander and Arblaster[29] provide some support in relation to fire potential, mentioning that there have been significant increases in hot days and warm nights and decreases in cool days and cold nights over the past few decades. Perkins et al. [30] claim that climate change, especially global warming, is to blame for increases in warm nights and hot days in numerous areas. In addition, Preston et al. [25] report that investigations of fire weather in southeast Australia have found support of rising average temperatures, rainfall reduction, and wind speed increases in correlation with increasing bushfire risk.

Limitations and future

Sharples et al. [24]state that the FFDI used in Australia were developed without the consideration of extreme fire weather. This implies that their use in predicting mega-fires, catastrophic fires and bushfires in extreme fire weather conditions are inadequate. In fact, recent study and research have established that FFDI is insufficient for predicting moderate intensity bushfire behaviour along with stronger intensities [24]. There are unquantifiable factors including fuel types and topography which can be crucial in determining fire behaviour and potential. As determinants of fire behaviour, certain fire danger rating systems can be lacking due to the absence of these factors. Australia's fire danger rating system is also limited in their capacity of predicting catastrophic of mega-fires, which can also be said for various other systems. Models observing current weather trends to predict future temperatures use different methods of determining these projections, making it difficult to select the correct one. Since the majority of models agree on future predictions of extreme temperature increases such as increasing warm nights, hot days, heat wave durations, dry days and decreasing cold weather, employing a single model in determining the likelihood of future bushfires is risky. Certain projections may overestimate certain trends and so it is important to use various projection models as to not generate unrealistic dangers upon the public, although informing them of bushfire occurrence probability is still vital.

With temperature and weather playing an essential role in bushfire potential, increases in global climate will definitely influence their behaviour. It is generally agreed that upon observing current global climate trends and using models to project future temperatures, warmer weather is expected for the future, generating a relationship between these changes and bushfire potential. Therefore, analysis of climate projections along with current temperatures will be needed to accurately determine their effect on the probability of bushfire ignition. Analysing climate models in regards with probably density functions and tail skill based measure is also a more accurate evaluation of climate models, able to discriminate between performances of models [30]. Using current fire rating systems including the FFDI, FWI and KDBI collectively with these future trends, we can find a general probability of bushfire occurrence. As each system is somewhat unique in their own way of utilising variables and factors, averaging out the potential fire danger ratings of each system can be done to get a general expectation of future bushfire occurrence. As a result, we can also possibly predict the occurrence of mega-fires in the future. Improvement on the precision of FFDI is feasible when new additional data is collected, able to be incorporated by the fire danger index to improve its accuracy and as well as precision [31].

FAILURE OF BRIDGE STRUCTURES

This section of the report looks at failures of bridge structures in more depth. It is organised according to the natural hazards. It covers both International and Australian examples of failure of specific road structures but also addresses the failure of some key materials such as steel, concrete and timber.

Wardhana and Hadipriono[32] categorized hydraulic causes of recent bridge failures in the US as flood, scour, debris, drift and others, which in total had 52.88 percent of the all reported bridge failures. However, the paper states that the flood and scour causes might have been used interchangeably by the data operators. Fire and earthquake causes of failure of bridges contributed 3.18 and 3.38 percent of the bridge failures respectively. Diaz et al. [33] investigated common causes of bridge collapse in Colombia. Considering only structural failure causes on the bridges in Colombia, most common failures are due to scour (35%) and riverine flooding and avalanches (35%) (Figure 13), which have caused total, partial and embankment damages [33]. Scheer[34, 35]compiledbridge failure cases from various counties including Australia, Germany and England. The partial or total failure of bridges due to fire and flooding have been recorded. Although there is no categorization of failure mechanisms mentioned in [34, 35], 1. spalling, 2. cracking, 3. impairmentof concrete and reinforcement were the main failures in concrete bridges; total and partial burned and damaged timber bridges for timber bridge failures;1. excessive deflections of web and 2. bottom flange of the girders for steel bridges were mostly identified.

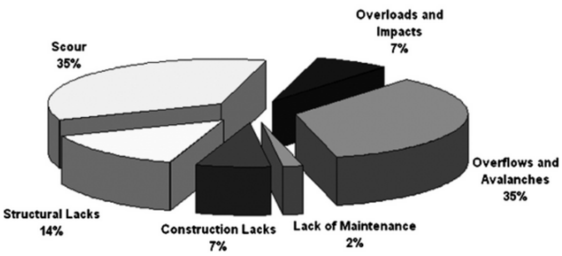


FIGURE 13: CAUSES OF BRIDGE COLLAPSE IN COLOMBIA[33]

Cook [36]analysedbridges failure in the US and stated that 52.17% of bridge failure are due to hydraulic causes and 3.26% are caused by fire. Cook mentions that the number of hydraulic failures on the bridges which were constructed over water have been much higher than the bridges built over roadways & railroads (379 to 7 respectively). This difference is 12 to 6 for failures caused by fire (bridge built over water and over roadways & rail roads respectively).

Cause of Failure	Partial Collapse	Total Collapse	Total Count	Percentage of Total
Hydraulic Total	21	27	48	52.17%
Hydraulic	–	2	2	2.17%
Flood	8	18	26	28.26%
Scour	12	7	19	20.65%
Ice	1	–	1	1.09%
Collision Total	17	1	18	19.57%
Collision	14	1	15	16.30%
Auto/Truck	3	–	3	3.26%
Overload	3	8	11	11.96%
Deterioration Total	4	2	6	6.52%
Deterioration	–	1	1	1.09%
Steel deterioration	2	1	3	3.26%
Concrete deterioration	2	–	2	2.17%
Fire	3	–	3	3.26%
Construction	1	1	2	2.17%
Fatigue-steel	1	–	1	1.09%
Bearing	–	1	1	1.09%
Soil	1	–	1	1.09%
Miscellaneous	1	–	1	1.09%
Total	52	40	92	100.00%

FIGURE 14: PERCENTAGE OF CAUSE OF BRIDGE FAILURES [36]

FLOOD

Failure Mechanisms

There are many ways that a bridge could be damaged in an extreme flood event. If the structure is completely inundated during the flood, the damage to the property depends on the length of time it was submerged as well as the elements collected around or passing the structure. Even after the flood water recedes, extra care should be taken to inspect the supports of the bridges. 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 (Figure 15). Movement of abutments, wing walls and piers, rotation of piers and missing, damaged dislodged or poorly seating of the bearings are the major reasons for substructure failure. Superstructure could be damaged due to the debris on deck, rotation of deck, dipping of deck over piers or damage of girders. Due to any of these reasons, the members of a bridge could be damaged and bridge may not be completely functional.

Gourlay in [5] states a number of factors which are affecting the damages on bridges due to floods in Australia. The factors are: (a) lack of hydrologic data upon which to base estimates of the magnitude of floods for design purposes; (b) ignorance of the hydraulics of flow in alluvial rivers and flow through bridge waterways and around bridge piers; (c) lack of reliable methods for estimating scour at bridge piers assuming that adequate information is available concerning flood flows and scour resistance of bed materials; (d) inability to predict the occurrence of impact and/or the accumulation of debris against the bridge structure[5].

FIGURE 15: GRANTHAM FLOOD - COURTESY OF DAILY TELEGRAPH

The Lockyer Valley Region of Queensland and the floods that occurred in 2011/2012 had a grave effect on the bridge infrastructures which severely impacted the surrounding communities. The Lockyer Valley Regional Council has assembled bridge inspection reports for forty-seven bridge structures in the region which were adversely affected by the floods. These comprehensive details relating to each of the affected bridges will provide the methodological foundation for identifying the particular attributes of bridges contributing to failure such as bridge surface, bridge substructure, bridge superstructure, bridge approaches and waterway as well as recognising vulnerable bridge structure elements. This highlights the importance of examining and analysing the ramifications of isolated member behaviour on the study of the performance of a structural system in its entirety.

Sinclair Knight Merz (SKM) – a private organisation prepared a bridge inspection template in accordance with the Queensland Transport Main Roads Level One bridge inspection in order to observe and verify the evaluation for each inspected bridge. The following data was successfully compiled:

- Bridge Surface
  - Footpaths – impaired
  - Barriers – damaged including lost fixings, unfastened post base
  - Bridge surface – damaged or missing, scuppers clogged
  - Expansion joints – loose and in need of repair, obstructions in gap
- Bridge Substructure

- Abutments
  - Movement of abutments
  - Disintegration and erosion of spillthrough
  - Shifting of the wing walls
- Bearings
  - Missing, damaged or dislodged
  - Poorly sealed
- Piers
  - Migration of piers
  - Scour around piers
  - Rotation of piers

➢ Bridge Superstructure
- Girders
  - Impaired
- Deck
  - Damaged and in poor condition
  - Debris found aboard the deck
  - Dipping of deck over piers
  - Rotation of the deck

➢ Bridge Approaches
- Signs – swept away, impaired or obscured
- Road surface – lost or damaged, settlement
- Guardrails – impaired or missing
- Road drainage – clogged inlets and outlets

➢ Waterway
- Scour protection run-down
- Scour punctures
- Bank erosion
- Debris build-up against substructure
- Debris build-up against superstructure

Oh et al. (2010) explains that the vulnerability of a structure is dependent on its physical attributes such as the type of material and construction practice utilised, the bridge's height and elevation. From observing the SKM reports, an underlying notion that different bridges have different types of failure mechanisms is highlighted. This accentuates the significance of analysing the consequences of individual member behaviour for the purpose of obtaining performance based data of an entire structural system[37]. The investigative reports on the bridges affected within the Lockyer Valley region illustrate that some bridge infrastructure failed due to loss of bridge approach while others failed because of scouring at the bridge pier or abutment. More details of this case study are given in the Case Study section of this report. A table portraying the failure mechanisms of different bridge structures underlines the vulnerabilities of bridges during flood events (Table 3).

TABLE 3: INVESTIGATED FAILURES OF BRIDGES IN LOCKYER VALLEY [38]

Name	Category	Underwater?	Mode of Failure	Most affected bridge component
Peters	4 Span Precast Concrete Deck	Yes	Abutment headstock movement results in loss of connection to piles; Headstock not centrally positioned on piles; Run on slabs have been debilitated; Cracking of piles	Both run on slabs/scouring or debilitated
Davey	2 Span Blade pier R/C vertical abutments	Yes	Considerable crack in western wing wall; Guardrail impaired due to build-up of debris; Substantial scouring of western abutment	Abutment wing/wall – scoured and cracked
Logan	4 Span Blade pier R/C vertical abutments	Yes	Damage to one whole approach section; Substantial scouring of eastern abutment; Debilitation of headstock; Cracks within eastern abutment	Bridge approach and abutment scouring
Sheep Station	Single span precast deck unit	No/Medium	Abutment wing wall dropped and rotated causing large cracks; Wing wall not linked to the headstock; Western spill through undermined	Abutment wing walls either scoured or undermined
Murphy	Concrete Deck Unit	Yes	Substantial debris build-up on the deck; Approaches from northern direction scoured with road surface and pavement undermined	Bridge approach scouring
The Dairy	2 Span timber girder – concrete deck	Yes	Removal of rip rap spill through scouring protection with slight undermining of abutment headstocks	Abutments scoured or undercut
McGrath	3 Span Deck Unit bridge	Yes	Leaning of pier supporting pile; Pile not centrally positioned on the headstock; Erosion and scouring identified beneath both back spans	Approach embankments – eroding and scouring

Maggarigal	2 Span Deck Unit (Precast Concrete)	Yes	Bridge approach and deck substantially impaired; Build-up of debris and mud on the structure and approach	Bridge approach and deck scoured and debilitated
Middleton	4 Span Timber Deck	Yes	North abutment undergoing scouring; South abutment undermined	Abutments - scouring
Belford	2 Span I girder bridge	Yes	Significant cracks developing in the western wing wall; Southern upstream rock spill experiencing scouring; Approach road and relieving slab undercut	Wing wall and abutment scoured and undercut
Frankie Steinhardt's	Single Span precast concrete bridge	No/Medium	Substantial scouring occurring at the approach embankments on opposite corners of the bridge; Approach embankment is unstable and tension cracks are forming on the pavement	Embankments from both approaches – scouring
Duncan	4 Span Deck Unit	Yes	Missing road shoulder at the end of the bridge; Formation of scour holes on eastern abutment	Abutment and bridge approach scouring
The Willow	Single Precast Deck Unit	Yes	Guardrails removed; Upstream edge of the structure is impaired; Both approaches heavily damaged	Both bridge approaches – scoured
Greer	4 Span timber girders with concrete deck	No/High	Scour protection has been worn away from the spill through surface; Spill through heavily scoured	Spill through –scouring

As highlighted by the table above and the inspection reports undertaken for the bridges affected by the 2013 flood events in Lockyer Valley, different bridges undergo varying failure mechanisms. The major failure mechanisms detected are:

- Bridge approach and deck were substantially impaired
- Abutment and pier scouring
- Undercut run on slabs on both sides of bridge structures
- Substantial build-up of mud and debris on bridge approaches and structure
- Abutment headstock not linked to piles
- Significant cracking of the abutment wing walls

In the following sections, three major failure modes identified above: scour, debris and flood impact and lateral displacement of the superstructure leading to displacement of structural elements are discussed in detail.

Scour – Effects, Predictions, Models and Countermeasures

What is Scour

Scouring has been recognised to be the most prevalent cause of bridge infrastructure failure in the United States[39]. It is the result of erosive processes of flowing water, which undermines and carries streambed materials from around the foundations of bridges. Bridge scour can be defined as a dynamic process that produces altering outcomes dependent on factors such as flow angle and strength, water depth, pier and abutment size characteristics, material attributes of the soil sediment, and so on. The scouring phenomenon can be categorised into three types: local scour, contraction scour, and degradation scour – all of which affect the performance and structural integrity of bridges[40].

Local Scour: Local scour involves the removal of soil materials from around bridge foundations (piers/abutments) and is caused by an increase in acceleration of water flow and resulting vortices induced by flow obstructions such as piers or abutments.

Contraction Scour: Contraction scouring in a waterway involves the erosion of soil materials from the bed within all or most of the bridge reach’s channel width as a result of increased shear stresses and water flow velocities due to the narrowing of the bridge reach, enhanced discharge in the reach, or both. Contraction scour can be cyclic and is related to the passing of a flood.

Long-term Degradation Scour: Degradation and aggradation scouring refers to long-term streambed elevation modifications brought around by man-made or natural cause which can affect the bridge reach of the river of which the bridge is located. Aggradation entails the deposition of material scoured from the upstream channel of a bridge whilst, degradation involves the lowering of a streambed over relatively long reaches due to a deficit in sediment supply from upstream.

Although scouring can take place at any time, it is extremely prevalent and powerful during flood events. The process of scouring can result in bridge failures by creating structural instability particularly through their foundations. This highlights an apparent and discernible vulnerability of bridge structures during flood events and must be recognised to ensure the sustainability of future road infrastructure.

The Schoharie Creek Bridge Study

This case study along with a variety of journal articles were utilised for the methodology and approach to understanding scouring and procedures to minimise bridge vulnerability during flooding. The Schoharie Creek Bridge collapsed on the morning of April 5th, 1987 during the spring flood[41]. The bridge's failure was initiated by the collapse of pier three and within two hours all spans and piers gave way.

Wiss, Janney, Elstner (WJE) Associates along with Mueser Rutledge Consulting Engineers were assigned the task of investigating the bridge failure along with a number of other firms. Both teams concluded that the failure of Schoharie Creek Bridge was caused by widespread scouring under pier three. Furthermore, the vulnerability of scouring under this particular pier was affected by four significant factors[42]:

1. The utilisation of shallow footings bearing on soil was not enough to ensure that they would not be below the possible scour limit.
2. The piers foundation had bearing on erodible soil allowing the high flow velocity of water to penetrate the 'bearing stratum.'
3. The footing excavations and backfill had no resistance to scour, caused by the filling of erodible soil for the backfill; topped off with dry rip rap.
4. Riprap protection, inspection and maintenance management were inadequate

Although scouring was identified as the main case of bridge failure, several other factors were examined which may have contributed to the collapse of Schoharie Bridge. These factors include the design of the superstructure, the quality of materials used in construction, maintenance of the bridge and inspection of piers according to set guidelines, however, it was concluded that none of these items were the reason for the collapse of the Schoharie Creek Bridge[42].

Predicting Scouring

Bridge engineers and researchers alike have determined that scouring can be correlated to a number of elements such as: the dynamic hydraulic properties of water flow, the geometry of bridge piers/abutments and the physical characteristics of the channel. Predicting scour utilises the accessible data of these elements prior to a flood event in the hope of minimising potential scouring susceptibilities and therefore preventing structural failures of bridges.

Scour prediction practices can be classified into two distinct divisions: prediction through empirical equations and through neural networks.

Scour Prediction: Empirical Equations

Scour prediction through empirical equations have been widely studied and developed by a number of researchers [43]. The majority of equations established were related to laboratory results and field data – with differences between the two methodologies being factors that were considered during the development of scour models, parameters utilised in equations and laboratory/on-site conditions. These equations include[44]:

$d_s = 2.0 y K_1 K_2 K_3 (b/y)^{0.65} F^{0.43}$ (Federal Highway Administration)

Where:

- d_s is scour depth
- y is flow depth at the upstream of the pier
- K_1, K_2, and K_3 are correction factors for the pier nose shape, angle of attack flow and bed condition, respectively
- b is pier width
- F is Froude's number

The above equation which is promoted by the US Department of Transportation's Hydraulic Engineering Circular No. 18 HEC-18, was developed from laboratory data and is recommended for calculations associated with both live-bed and clear-water conditions.

Melville and Sutherland [45]are also one of many researchers who have formulated empirical equations for the prediction of bridge scour. Their equation is as follows:

$d_s = K_I K_d K_y K_a K_s b$

Where:

- d_s is scour depth
- K_I is flow intensity factor
- K_d is sediment size factor
- K_y is flow depth factor
- K_a is pier alignment factor
- K_s is pier shape factor

These and many other equations have been proposed and validatedby numerous researchers to ensure their accuracy when applied to field studies. Jones[46] is one such researcher who compared the established bridge scour equations using limited field data and laboratory data as tools, and was able to categorise all equations into three classes, namely, those of the University of Iowa, those of the Colorado State University, and those based on foreign literature. He discovered that the Colorado State University equation entailed the data, however, derived scour depths were less than other equations. Johnson [47] used comparative studies for seven of the most commonly cited scour equations utilising an extensive set of field data for both live-bed and clear water scour. Correlated differences between equations and certain limitations were explained in his study.

Landers and Mueller[48]analysed specific pier scour equations using 139 measurements of local scour in both live-bed and clear-water conditions and from their computed and observed scour depths, they found that none of the selected equations could estimate the depth of scour for all conditions accurately. Mueller [49] himself undertook a comparative study of twenty-two scour equations utilising field data provided by the USGS[50]. His personal study revealed that the HEC-18 (shown above) equation was adequate for design calculations due to the fact it rarely under-estimated the measured scour depth, however, frequently overestimated the observed scour. Mueller employed 384 field measurements of scour from fifty-six bridges for the purpose of his study. Although concluding information gathered from various comparative studies by different researchers are quite contrasting, it is believed, established on the basis of conducted laboratory experiments and field tests, that the majority of formulated equations may overestimate the scour depth and are generally conservative[51].

Scour Prediction: Neural Networks

The systematic flow around bridges' foundations is incredibly arduous and it is difficult to establish a general empirical model to provide the accurate estimation for scour. Regardless of the complexities of the scouring process, researchers have found two other explanations for why existing methods do not produce accurate scour predictions. These include:

1. Site conditions are much more complicated and sophisticated in comparison to laboratory conditions.
2. The limited ability of traditional analytical mechanisms of statistical regression to select appropriate parameters used in formulae and to recognise the relationships between these parameters and associated responses.

The adoption of artificial neural networks (ANNs) by a number of researchers for the prediction of bridge scour has been successful in recent studies [52]. The fact that ANNs do not require specified physical correlations between bridge scour (the output) and various elements that affect bridge scour (the inputs) can be considered an essential advantage in their utilisation. Because of ANNs flexibility regarding the definition of relationships between bridge scour and these various elements, they have the capability to accurately estimate scouring data more effectively than traditional regression based methods [53].

Studies undertaken by Bateni et al. [54] adopted the use of ANNs and an adaptive neurofuzzy inference system (ANFIS) to predict both the time-dependent scour-depth and the equilibrium, employing a large quantity of laboratory data. For the application of their studies, two ANN models were implemented (a multi-layer perception using back-propagation algorithm and a radial basis function using orthogonal least-squares algorithm) to model equilibrium scour depth as a function of five variables: flow depth, critical flow velocity, pier diameter, mean velocity and mean grain diameter. Test results concluded that the multi-layer algorithm ANN model provided superior estimations of scour depth in comparison to the radial based algorithm, the ANFIS models and previous empirical approaches. Lee et al. [55] is also one such researcher who utilised the Back-Propagation Neural Network to predict scour depth around bridge foundations and found this method to be efficiently capable of doing so. The latest study undertaken by Zounemat-Kermani et al. [53]based its scour prediction investigation on the adoption of two ANN models: the feed forward back-propagation model and the radial basis function using orthogonal least-squares algorithm. Their numerical test results highlight the fact that ANN estimations are more adequate than those obtained through empirical equations due to the low errors and high correlation coefficients. Implementing a sensitivity analysis further indicated that pile diameter and pile spacing to pile diameter parameters are the two most essential factors that influence scour depth.

Scouring – Models

As mentioned earlier within the review, the complexity of the scouring process requires extensive numerical and laboratory models to ensure a developed understanding of scouring and its relation to bridge susceptibilities, particularly during flood events.

Numerical Models

For the purpose of achieving accurate and well-developed numerical models, most studies involved comparative research with laboratory models. The Fukuoka et al. [56]study of a three-dimensional numerical model provided satisfactory accuracy regarding the simulation of local scour around bridge piers and their solutions can be effectively correlated with the experimental results obtained from large-scale hydraulic models. Further comparative studies between Richardson and Panchang's[57] fully three-dimensional hydraulic model and Melville and Raudkivi's[58] laboratory observations were undertaken to replicate the conditions that occur during water flow at the base of a cylindrical bridge pier with scoured holes. According to the results obtained between the studies, both quantitative and qualitative factors were well agreed upon with any discrepancies attributed to the particular parameters chosen in the numerical model.

Numerical models have not only been compared to laboratory simulations, but to empirical equations to ensure justification of their results. Young et al. [59] is one such study which established a numerical model for clear-water abutment scour depth along with an independent three-dimensional finite element model. These simulations displayed agreeable relationships and when evaluated against the HEC-18 empirical equation, the researchers were able to conclude that the scouring prediction is overestimated by twenty-two percent. Studies undertaken by Kassem et al. [60]similarly developed a computational fluid dynamics model, FLURNT, to reproduce field data. These studies also used comparative methods against laboratory measurements which produced agreeable results. They were able to demonstrate that the HEC-18 equation substantially overestimates the scour depth by utilising their chosen models. Following introduces a number of formulae for calculation of scour depth.

The Modified Laursen by Neil (1964) Equation

Ds/b = 1.35 (H/b)^0.3 (Hafez, 2004)

Where:

Ds = Equilibrium scour depth

b = Obstruction width (pier width)

H = approach water-depth

Shen et al. (1969) Formula

Ds/b = 3.4 (Fo)^(2/3)(H/b)^1/3 (Hafez, 2004)

Where:

Fo = Froude number

The Colorado State University or CSU Formula (1975)

Ds/H = 2.2(b/H)^0.65 (Fo)^0.43 (Hafez, 2004)

Jain and Fischer (1979) Equations

For (Fo - FC) > 0.2 Ds/b = 2.0 (b/H)^0.65 (Fo)^0.43 (Hafez, 2004)

For (Fo - FC) < 0.2 Ds/b = 1.84 (Fo)^0.25 (H/b)^0.3 (Hafez, 2004)

Where:

Fc = Critical Froude number

Youssef I. Hafez' Analytical Equation

(DsH)^3 = (3tan$\varphi/((SG-1)(1-\theta))(1/(1-b/B)$^2)(η2$Vx2/gH$)(1+ Ds/H)

Where:

SG = Sediment specific gravity

φ = Bed material angle of repose

ϴ= Bed material porosity

η = Momentum transfer factor

Vx = Longitudinal flow velocity of the jet attacking the bridge in the direction normal to the pier

B = Pier centreline to centreline distance in case of multiple piers

Laboratory Models

The notion that laboratory models regarding bridge scour can not only aid in understanding of distinct variables and parameters as well as improve scour countermeasures highlights essential advantages with the utilisation of such studies. This concept has motivated the development of a large-scale research onlaboratory models in the past two decades for analysing bridge scour.

The Hydraulics and Sediment Research Institute in Cairo [61] has led laboratory simulation studies for the Imbaba Bridge across the Nile, utilising a series of clear-water scour tests to investigate the principles of local scour against circular foundation piers. Concluding results determined that the observed scour hole of the pier was produced by incompatible velocity fields at the wake vortex stream crossroads from adjacent piers and by the confluence flow. This analysis resulted in the formulation of an empirical equation to estimate the confluence and wake maximum local scour depth of the described conditions. Umbrell et al. [62] correspondingly examined the clear-water contraction scour resulting from the pressure flow underneath a bridge without the presence of piers or abutment with the aid of tilting fume. A simulation of a variety of laboratory-controlled pressure-flow conditions against a model bridge deck was undertaken with a studied focus on factors such as: sediment size, approach velocity and pressure-flow velocity under the bridge deck. Laboratory tests involving a variety of water depths and flow velocities with two distinct consistent 'cohesionless' sediment diameters and a circular pile were utilised by Sheppard and William [63] to analyse local-clear water and live-bed scouring. With the aid of a tilting flume, the scour depth as a function of time is observed with acoustic transponders and video cameras. Assisted by their instrumentation, large bed forms were monitored to shift through the scour hole during a number of live-bed scour tests, concluding that Sheppard's formulations in[64] performed well regarding the range of conditions covered by the experiments.

Scouring – Countermeasures

Scouring mitigation has been a topic of much deliberation, with extensive techniques, measures and practices accessible for counteracting bridge scour at piers and abutments. Studies related to such mitigation can ensure a reduction in bridge vulnerabilities particularly during flooding. Countermeasures can be classified into two sections: armouring countermeasures and flow-altering countermeasures. Armouring techniques achieve a reduction in scouring without modifying the hydraulics of approach flows whereas flow-altering approaches aim at altering the hydraulic factors related to flows by utilising spur dikes, guide banks, parallel walls and collars to ultimately lessen scouring at bridge foundations. An extensive evaluation of varying countermeasures for bridge scour foundations can be found in Lagasse et al. [65] and Barkdoll et al. [66]. The table below highlights comparative working principles, advantages and problems regarding the two methods of scour countermeasures.

TABLE 4: COUNTERMEASURES METHODS

	Armouring Countermeasures	Flow-Altering Countermeasures
Principle	Armouring layer ensures protection of bed	Modify flow configuration or break-up vortices to

	sediments underneath from being scoured.	minimise scouring.
Advantage	Most frequently used technique; easy to use; adaptable to most situations.	A range of designs can be developed to suit varying site conditions for the achievement of adequate results.
Problems	Flailing of soil materials through the armoured layer; difficult to keep armour in place; narrows the channel which results in further contraction scour.	Special design might be required for specific conditions; substantial costs associated with the construction of new structures when required.

The mot frequented method of armoured scour protection is riprap[67] with other armouring techniques including tetrapods, cable-tied blocks grout filled bags, mattresses and concrete aprons. Paker et al. [68] conducted numerous studies involving the use of models, laboratory tests and experiments to investigate the use of riprap as a scour countermeasure. Their extensive reviews demonstrated that the optimum positioning of riprap was at a depth below the average bed level. The experimental studies of failure mechanisms, stability and placement positioning of riprap at bridge piers undertaken by Lauchlan and Melville[67] verified the analysis of Parker et al. [68] with their results conforming to the notion that the deeper the placement of riprap, the better protection against local scour it provides.

Flow altering countermeasures such as the use of: submerged vanes [69], sacrificial sill [70], collars and slots[71] and parallel walls [72] have been studied to achieve the minimisation of bridge scouring. Most of these mitigation techniques utilise devices on the upstream side of bridge piers or by modifying the geometry of the piers facing the approach flow. Depending on the nature of the scouring issue – local scouring at bridge foundations, contraction scouring across the bridge opening bed, channel degradation or lateral channel movement, a particular countermeasure must be selected [73]. Some cases even require the utilisation of countermeasure 'bundles' to enhance scour protection. Comparative studies of varying countermeasures were undertaken by Lagasse et al. [74] in regards to the form of scour, hydraulic condition and maintenance – providing design guidelines for different techniques.

Countermeasure uncertainties relating to a lack of systematic testing and unidentified impending failure has resulted in studies such as those developed by Johnson and Niezgoda[73] whereby a risk-based method utilising effect analysis, failure modes and risk priority numbers are introduced for comparing, ranking and selecting the most adequate scour countermeasure. The arising uncertainties in their studies wereintegrated within the failure modes while effect analysis was incorporated in the selection process consisting of risk in terms of the consequence of failure, the amount of difficulty needed to detect failure and the possibility of a component failure. The resulting risk priority numbers are utilised to provide validation for the selection of a particular countermeasure.

Kapernicks Bridge, Lockyer Creek (case study)

Kapernicks Bridge is a three span, two lane precast concrete Girder bridge located on Flagstone Creek Road. The superstructure comprised of prestressed concrete I girders with a reinforced concrete composite slab. Kapernicks Bridge has two piers on each side of upstream and downstream. Tragically, bridge was failed due to the debris impacts arisen from the flood during January 2011. The failure of bridge occurred due to an accumulation of neglected structural shortcomings and the scour under the piers. Even though, the bridge had severe damages by the flood, it had now been repaired and is now in service. Prediction ofthe bridge pier scour depth ofKapernicks Bridge is made in terms of upstream and downstream sides[75].

FIGURE 16: KAPERNICKS BRIDGE AFTER THE FLOOD ON JANUARY 2011 (BY LVRC)

Upstream of Kapernicks Bridge

The upstream view of the Kapernicks Bridge is shown in Figure 17with input parameters neededfor the prediction of the scour depths ateach pier which are local bed level, local water depth, centerline depth, local velocity, pier width and soil parameters. Prediction using the equations was carried out with some of the assumption for the unknown information. Due to the lack of given information of the bridge, the average approach flood flow velocity was assumed to be 3m/s, sediment under the bridge was assumed to be sand, and channel width was assumed to be 4m.

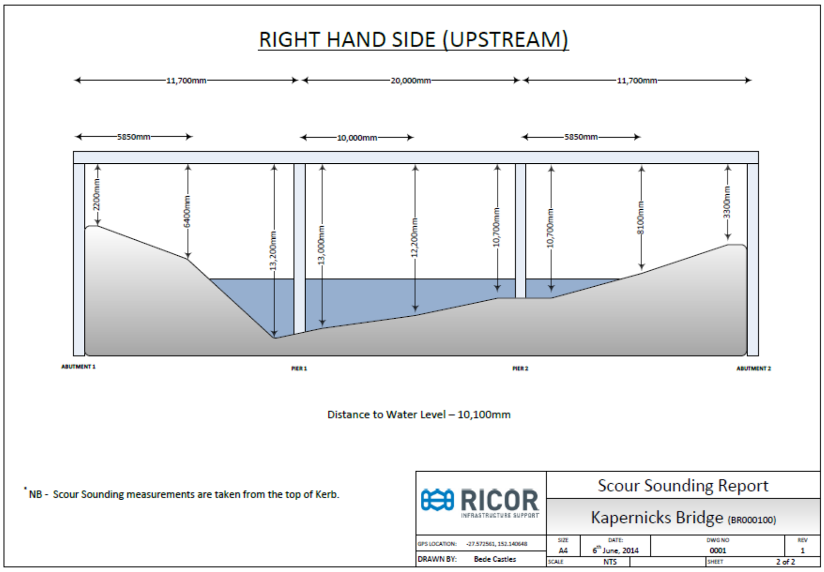


FIGURE 17: KAPERNICK BRIDGE UPSTREAM

Calculations of bridge pier scour depths of Kapernicks Bridge were carried out under various flow velocities from 2m/s to 5m/s. Initial calculation was carried out under the local velocity of 3m/s independently and it was noted that the scour depths derived from Youssef I. Hafez' Analytical Equation was 0.169m. This much of magnitude will not lead to collapse of the bridge. A sensitivity analysis was conducted by changing the water velocity. As shown in Table 5 and Table 6, scour depth increase with thelocal velocity.. However, the scours depths occurred on Pier No.2 are relatively lower than Pier No.1 because of relatively low local water depth compared to the Pier No.1. It is concluded that scour was not the main cause of collapse of the Kapernicks Bridge in 2011.

TABLE 5: TABLE OF AVERAGE BRIDGE PIER SCOUR DEPTHS (M), UPSTREAM

Average Bridge Pier Scour Depth (m) of Upstream				
	Local Velocity, Vc, 2m/s	Local Velocity, Vc, 3m/s	Local Velocity, Vc, 4m/s	Local Velocity, Vc, 5m/s
Pier No.1	1.899	2.600	3.269	3.918
Pier No.2	0.126	0.169	0.208	0.245

FIGURE 18: GRAPH OF AVERAGE BRIDGE PIER SCOUR DEPTHS (M), UPSTREAM

TABLE 6: AVERAGE BRIDGE PIER SCOUR DEPTHS (M), DOWNSTREAM

Average Bridge Pier Scour Depth (m) of Upstream				
	Local Velocity, Vc, 2m/s	Local Velocity, Vc, 3m/s	Local Velocity, Vc, 4m/s	Local Velocity, Vc, 5m/s
Pier No.1	0.914	1.241	1.549	1.847
Pier No.2	1.238	1.687	2.111	2.522

FIGURE 19: GRAPH OF AVERAGE BRIDGE PIER SCOUR DEPTHS (M), UPSTREAM

Failure of superstructure

Failure of bridge super structure due to water flow, debris impact and log impact is discussed in this section. Failure mode could be local damage to the superstructure or displacement of the structure. Forces are calculated using basic principles of energy, based on assumptions on the size of debris and log. A major issue identified in recent failure case studies is the urban debris, which is dissimilar to a typical log impact allowed in the Australian standards.

Forces on the superstructure due to water flow

Where the superstructure is partially or fully submerged in the flood, the design horizontal drag forces on the superstructure, normal to its longitudinal axis, may be calculated as:

$$F_{d=}\ 0.5C_dV_d^2A_d^2$$

Where V_d = mean velocity of water flow at superstructure level

C_d = drag coefficient

A_d = projected area of the superstructure (including any rails or parapets) normal to flow in (m^2).

Typical velocities encountered during the Lockyer Valley floods were recorded as up to 4 m/s.

Debris impact

Waterborne debris composed primarily of tree trunks and limbs often accumulate on bridges during flood events[76]. Debris accumulation is a significant problem at bridges because it tends to exacerbate both flooding and scour around the bridge foundations, as well as loading on the structure. It is critical at the time when the debris accumulates on bridges during flood events. The seriousness in terms of debris impact onto bridge pertains to the magnitude of a flood event and the concentration of debris within floodplain. Flood debris may contain vegetation, trees, mud, soil, sediment, damaged structures, vehicles, food waste, etc. (Hickenlooper et al, 2013). Woody debris from upstream areas of forested or wooded watersheds is often transported to streams during heavy rainfall events. If the debris reaches a bridge pier, it may be caught and accumulated on the pier, effectively narrowing the waterway opening. As debris continues to accumulate during subsequent high-water events, problems of flooding, scour, and loading on the pier are often intensified. In some cases, the accumulated debris can block most or all of an entire span. Woody debris can obstruct more than 50% of the water channel of the bridge [77]. The accumulation of debris cause superficial damage such as spalling of concrete from piers. Moreover, as in hydraulic damage, the pressure of water due to the river/stream flow and debris accumulation, results in overturning of bridge from the supports and buckling failure of the substructure (Fenske et al, 1995). The extent of damages of bridgees due to debris impact depends on the characteristics and supply of debris transported to bridge. The effects of debris accumulation are varying with the range from minor flow constriction to severe flow contraction resulting in significant bridge foundation scour.

As discussed earlier, AS5100.2 recommends the following formula for calculating the forces due to debris on bridges.

$F_{deb}= \ 0.5\, C_d V_u^2 A_{deb}$

Where;

A_{deb} = projected area of debris

V_u = flood velocity

C_d = drag coefficient

A typical calculation for the debris impact is shown below.

Debris Forces (Cl. 15.5.1 AS 5100)				
Depth of Debris	=		3m (cl. 15.5.1)	
Lenghth of Debris	=		20m (cl. 15.5.2)	
Ad =	60			
Cd =	1.04			
V =	2.32			
F(deb)=	0.5*Cd*V^2*A			
	167.9309	kN		

Log impact

Where floating logs are possible, the serviceability and ultimate design forces exerted by such logs directly hitting piers shall be calculated on the assumption that a log with a minimum mass of 2 tonnes shall be stopped in a distance of 300mm for timber piers. Should fender piles or sheathing to absorb the energy of the blow be placed upstream from the pier, the stopping distance may be increased. The design forces shall be calculated using the mean velocity of water flow.

The forces due to log impact and debris shall not be applied concurrently. Log impact shall be applied with such other water flow as are appropriate.

Lateral displacement

Forces from the flood water flow, debris and log impact increase the possibility of lateral movement of bridges. Hydrostatic force of flood water also helps uplifting the bridge deck and facilitates the occurrence of lateral displacement. This failure mode can be simulated by considering a combination of the flood loading calculated to be applied on the structure.

Lateral displacement of the bridge towards downstream side

FIGURE 20: LATERAL DISPLACEMENT DUE TO FLOOD [78]

EARTHQUAKE

Bridge performance during earthquakes

Bridges are the main structural system of transportation systems. Although new bridges are designed with improved seismic design guidelines, existing bridges are more vulnerable for failures due to earthquakes. There is lack of available information on structural damages specific on bridges from the historical data mentioned previously in Australia which prompts the consideration of other earthquake cases from other countries. The behaviours and failure mechanisms of these bridges will be taken as a reference to develop understanding and be able to identify potential bridge elements that are at risk in Australia. 1989 Newcastle, NSW earthquake with magnitude of 5.6 was considered as the largest destructive earthquake occurred in Australia in terms of property and life losses. Yang and Molloy [79] conducted a seismic assessments of the bridge over Spencer Gulf at Pt Augusta in South Australia and some retrofitting techniques were introduced. Past earthquakes such as 1989 Loma Prieta, 1994 Nortridge, 1995 Kobe and 2010 Chilie earthquakes have caused many damages to bridges around the world and following paragraphs describe the failure mechanisms of girder bridges due to these types of earthquakes.

Bridge design is carried out around the world similar to loading requirements are standards until 1972 [80]. In recent times, due to the high risk of earthquakes, some high seismic requirements were introduced in design of structures. Generally bridges are designed for frequent vertical loads and super structure is typically stiff enough to sustain during earthquakes and do not undergo inelastic deformations. From past failures of bridges in overseas, have shown that bridges generally perform well during earthquakes and do not require additional considerations other than deck joint separations and restraining super structure movement at support points. However bridge sub structures are seen to be more vulnerable to damage and most of the damages caused due to earthquakes in bridges are due to these types of failures [80]. Seismic performance of concrete girder bridges illustrates that there are some retrofitting techniques are needed during strong ground motions. Hwang et al., [81] evaluated a multi span girder bridge in transverse seismic action and observed that columns and bearings are more vulnerable to experience damage. Shinozuka et al., [82] conducted a similar assessment for the same bridge using nonlinear static analysis and drew the same conclusions. Using response spectrum analysis and linear time history analysis, Cancer et la., [83] conducted a study on simply supported concrete bridges expansion joints and they investigated on ways of retrofitting it.

Nielson and DesRoches[84] performed a three dimensional non linear time history analysis on a multi span typical concrete girder bridge in United States. They observed that reinforced concrete columns and abutments are more vulnerable to seismic damages. Also they noticed unseating of girders is a problem in concrete girder bridges due to seismic actions.

Reinforced concrete columns and prestressed girder in Uriage bridge in Japan was damaged due to 1978 Miyagi-ken-oki earthquake. The ends of the prestressed girders, where seismic forces, dead loads and anchor loads were concentrated were damaged again due to 2011 Tohoku-Oki earthquake [85]. Elastomeric bearings generally performs well in these types of strong ground motions [81, 85]. However some elastomeric bearings were failed due to liquefaction [85].

There are many examples which show different types of failure mechanisms of bridges in other countries during an earthquake event. Large number of structures are fractured and destructed due to the great Hanshin–Awaji earthquake and the reason for fracture of Nielsen bridge type bridge bearing was not clear initially. But later they have found that an impact due to relatively lower velocity between upper and lower bearings have generated stress which is sufficient to lead the fracture in upper bearing [86]. The catastrophic failures of bridges due to Chile earthquake in 2010 showed that important roles is played by soil liquefaction, settlement and embankment failures. Aspects such as shear failure of steel piles, shear failure of concrete substructure elements, failures and severe buckling of steel braces, failures of shear keys and restrainers at supports, and damage to girders due to lack of diaphragms were also common problems in some bridges [87]. The Tohoko-Oki earthquake caused lot of damages to bridges structures and these damages includes span unseating, column shear and flexural failures, approach fill erosion. Many past literature is available on failures of bridges due to earthquakes on both super structure and sub structure failures [88, 89]. In addition to the above mentioned failure mechanisms of bridges, collapses caused due to spans dropping off supports, separation of deck joints and collapse of suspended sections, columns pulling out of footings, shear in support column bases due to failure of confinement reinforcement etc in some important bridges due to past earthquakes [90]. The 2011 Great East Japan earthquake also caused lot of damages to bridges and similar types of failures were noted. Improving the confinement effect and shear capacity of the RC columns as per the post 1995 design guidelines and retrofitting techniques have improved the performance of bridges in Japan [91] .

(a) (b)

FIGURE 21: (A) FRACTURE OF BRIDGE BEARING OF NIELSEN BRIDGE TYPE DUE TO HANSHIN–AWAJI EARTHQUAKE

(B) LOSS OF SUPPORT AT ABUTMENT OF THE MANUEL ANTONIO MATTA BRIDGE DUE TO CHILE EARTHQUAKE

(C) LATERAL MOVEMENT OF ABUTMENTS DUE TO CHILE EARTHQUAKE

(D) SETTLEMENT OF BRIDGE APPROACH SLAB DUE TO CHILE EARTHQUAKE

Also vertical acceleration of earthquakes has caused many damages to bridges and in the design of structures especially made of concrete have shown unquantified errors in response predictions and unexpected damages due to vertical accelerations [92].

Bridge design in Australia

Past significant, recorded earthquakes in South Australia [93] were of a level between MM VII and VIII (TABLE 7), though the peak ground shaking usually was restricted to a relatively small area, perhaps affecting only a handful of bridge structures at most. Intensity contours for these SA earthquakes can be seen at the SA Primary Industries and Resources web site

TABLE 7: SIGNIFICANT AUSTRALIAN EARTHQUAKES

Event	Magnitude (&Mercalli Intensity)
1897 Kingston-Beachport	M 6.5 (MM VIII)
1902 Warooka	M 6.0 (MM VII, peak at VIII)
1954 Adelaide	M 5.5 (MM VII, peak at VIII)

1989 Newcastle (for comparison)	M 5.6 (MM VI, peak at VIII)

For a strength limit state earthquake (MM VIII) in Australia, bridges should perform to the following criteria:

• Depending on the importance of a structure, an earthquake of intensity MM VIII should be resisted without significant damage, and the bridge should behave elastically.

• Some structure(s) with low importance may sustain damage, which prevents the bridge(s) from remaining open to traffic.

Despite the rare occurrence of seismicity in Australia, the structural damages and consequences can still be devastating and costly. The performance of bridge structures are particularly important to ensure emergency responses are operational and accessible, which prompted the publication of the design standard for bridges in 2004. The design of bridges in response to earthquake loads is detailed in Section 14 of AS 5100.2:2004. This code makes a direct reference to the structural earthquake code in AS 1170.4 first published in 1993 which has been changed and updated since in 2007.Austroads technical report addressed these incompatibility issues and provided a provisional adjustments to the AS 5100.2 to adhere to the latest AS 1170.4:2007 [94].

The seismic design rules in Australian Standard for Bridge Design ,AS 5100 [95] were developed based largely on force-based design approaches. The seismic force level corresponding to elastic response to a design acceleration response spectrum for a soil site class is calculated based on an estimate of elastic stiffness of the structure. This elastic force is then modified by a Structural Response Factor, R_f, for an assumed ductility capacity of the bridge pier and an importance factor, I, for the expected performance in an earthquake. Current ASBD classifies bridges into three different types (Type I, II and III), which is similar to other international bridge design codes. Type III bridge is comparable with life-line / critical bridge in AASHTO[96], CAN/CSA-S6 [97] and EC8 [98]. Similarly, Type I and Type II bridges are comparable with emergency-route / essential bridge and other bridges, respectively. Importance factor (I) for Type I and Type II bridges is 1.0 and for type III bridges is 1.25. It is noted that I-factors suggested in ASBD are significantly lower than the recommendations in major bridge design codes [96-98]. Although in major seismic design codes expected performances of bridges in future earthquake events have been specified (Table 1), no such specification has been provided in AS 5100 [95]. It is believed that similar multi-level performance objectives should also be anticipated for the bridges designed for different importance levels according to bridge design standard [99]. The changes on the factors will ultimately influence the procedures in the determination of earthquake design forces. The changes on the response factors as well as the elimination of the ambiguities from the 2004 code edition will align the Australian bridge seismic code with the standard of the New Zealand bridge code, but with the consideration of low to medium seismicity to account for the appropriate earthquake category in the country [100].

In spite of these considerable differences, the general design procedure using the force-based approach is nonetheless still relevant. In the current bridge design code, two alternative analysis are provided for the determination of design forces, either using static or dynamic analysis. The overall bridge design steps are summarised on and (Standards Australia, 2004).

TABLE 8: BRIDGE EARTHQUAKE DESIGN CATEGORY (BEDC) IN AS 5100.2

Product of acceleration coefficient and site factor (aS)	Bridge Classification		
	Type III	Type II	Type 1
aS ≥ 0.2	BEDC-4	BEDC-3	BEDC-2
0.1 ≤ aS< 0.2	BEDC-3	BEDC-2	BEDC-1
aS< 0.1	BEDC-2	BEDC-1	BEDC-1

TABLE 9: BRIDGE DESIGN ACTIONS FOR EARTHQUAKES IN AS 5100.2

Bridge Category	Structural configuration and regularity	Method of analysis	Earthquake forces to consider
BEDC-1	Span ≤ 20m	No Action	N/A
	Span > 20m	Static Analysis	Horizontal
BEDC-2	Span ≤ 35m	Static Analysis	Horizontal
	Span >35m	Static Analysis	Horizontal and vertical
BEDC-3	One dominant mode of free vibration	Static Analysis	Horizontal
	More than one dominant mode of free vibration	Dynamic Analysis	Horizontal and/or vertical
	Complicated structures	Dynamic Analysis	Horizontal and/or vertical
	Irregular mass		
	Irregular stiffness		

BEDC-4	All bridges	Dynamic Analysis	Horizontal and/or vertical

The Austroads guideline identified the extent in which the AS 5100.2 needs to be clarified and adjusted in order to ensure consistency and compatibility in determining the BEDC from the latest AS 1170.4. These areas are (i) bridge level of importance, (ii) use of hazard factor, (iii) influence of structure height, (iv) subsoil class influence, and (v) bridge earthquake design category.

BUSHFIRE

Fire disaster impact on road infrastructure could be aresult of bushfires or accidents on the roads or on their adjacent environment. The impact of the elevated temperature caused by fire on material types used in construction of bridges, culverts and flood-ways could lead to degradation of structural or functional capacity of the structures and eventually failure of their elements.

FIGURE 22: FIRE - STEEL BRIDGE[101]

FIGURE 23: FIRE - CONCRETE BRIDGE [101]

Responses of structures exposed to fire can vary. However, these responses could be categorised in thermal, mechanical & deformation responses. There are typical mechanisms affecting structures due to fire on reinforced concrete, steel and timber structures (Figure 22 and Figure 23) which are categorised as follows:

Reinforced concrete

- Concrete spalling
- Concrete cracking
- Concrete delamination
- Compressive strength reduction
- Steel reinforcement and prestressed strands strength reduction

Steel

- Steel distortion
- Deflection of steel elements
- Formation of plastic hinges
- Buckling (web buckling)
- Reduction of tensile and yield strength
- Post-fire steel toughness
- Steel pitting & flaking
- Paint and coating degradation

Timber

Timber components of bridges can be ignited by the fire in the vicinity of the bridge. Whentimber is ignited the exterior layer starts to char. On one hand, charring causes the reduction in the strength of the timber element and on the other hand, it insulates the timber core which prevents the excessive temperature to reach the core of the element. Therefore, the core of the element will lose strength due to high temperature. The following material properties affect the timber elements of bridges in bushfires[102]:

- Charring (charring rate)
- Strength loss
- Elasticity loss

Following sections describes some mechanical properties and failure mechanisms of steel and reinforced concrete as main construction material components of bridges due to fire exposure.

Elevated temperature effects on failure

This section explores the local and global effects of the elevated temperature on concrete and steel materials and bridge structures which increases the vulnerability of the bridges due to bushfire events and can lead to partial or total failure of bridges.

Steel Yield strength

Astaneh-Asl et al.[103] states that the yield strength gradually decreases as the temperature increase in steel components. However, once temperature passes 500ºC the yield strength will decrease more rapidly (as it can be seen inFigure 24). Furthermore, it has been observed that the yield strength of steel at 530ºC drops to approximately 50 percent of its yield strength at room temperature, 20ºC. It is also described that the approximate 50 percent drop of steels yield strength and its further decrease as temperatures rise eliminates the safety factor applied for steel bridges (the safety factor usually being 1.5 to 2.0) thus may result in failure of the components. Garlock et al.[104] have stated that steel undergoes a phase change approximately about 721°C and if the temperature does not exceed that, one can assume that there was no great influence on the mechanical properties, including yield strength, of the steel. Kodur et al.[105] used different models to obtain yield strength of steel at elevated temperatures and it concluded that yield strength decreases as temperature increases. This conclusion was based on the reasoning that the nucleus of the iron atoms in the steel move farther apart as temperature increases in steel leading to decreased bond strength and hence lower yield strength. Chen et al. [106] present the mechanical properties of high strength structural steel and mild structural steel at elevated temperatures in their literature using an experimental program where steady and transient-state test methods were conducted at different temperatures. This literature concluded that both high strength steel and mild steel have similar reduction factors of yield strength between temperature range of 2°C to 540°C however differing reduction factors of yield strength for temperatures above 540°C. This literature also concluded that as temperature increased both high strength steel and mild steel had lower yield strength. To sum up, although the literatures agree upon the yield strength of steel decreasing as the temperature increases however there are differing evidence onthe probability failure of steel components between approximately 530°C to 721°C.

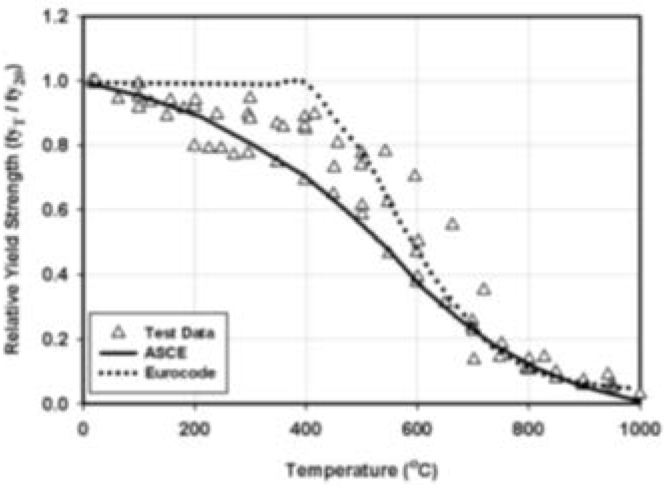


FIGURE 24: REDUCTION IN YIELD STRENGTH OF STEEL AT ELEVATED TEMPERATURES[101]

Steel Modulus of Elasticity

Wright et al.[101] states that the elastic modulus of steel decreases as temperature increase (as presented in Figure 25 below). This literature described the variations in elastic modulus to be due to the differences in steel grade and experimental techniques used. As explained for yield strength, Kodur et al. [105] reasoned the decrease in elastic modulus of steel with increase to temperature to be associated with the nucleus of iron atoms moving farther apart as temperature rises in the steel. Studies of Chen et al. [106] also present that there is reduction in relative elastic modulus of steel as temperature increases. To sum up the articles reviewed elastic modulus of steel decreases as temperature increases.

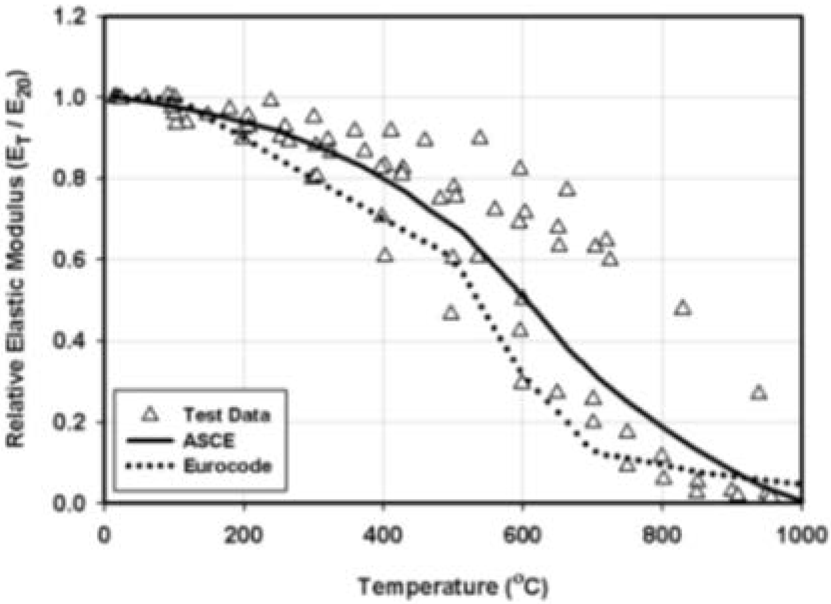


FIGURE 25: REDUCTION IN ELASTIC MODULUS OF STEEL AT HIGH TEMPERATURES[101]

Stress-Related Strain in Steel

The literature suggests that as temperature increases steel loses strength [107]. Figure 26 shows the stress-related strain curve for typical hot rolled steel to decrease as the temperature increases. Wright et al. [101] explains that besides the changes in yield strength and elastic modulus, the exposure of steel to elevated temperatures results in changes in its stress-strain curve shape. This literature describes the behaviour of steel at room temperature to be close to elastic-perfectly plastic, however as the temperature increases the shape of the steel stress-strain curve is expressed to become more rounded with a large decrease in the proportional limit. El-Rimawi et al. [108] have also concluded that the increase of temperature decreases the stress-strain curve. This literature explains that the drop in the stress-strain curve, due to elevated temperatures, could cause surpass in the elastic limit in which case permanent deformation will be experienced by the steel component, whereas before reaching the elastic limit the steel component may be deformed however it would be temporary. Buchanan[107] states that the stress-related strains in structures exposed to fire is possible to be greater than the yield levels and thus causing considerable plastification.

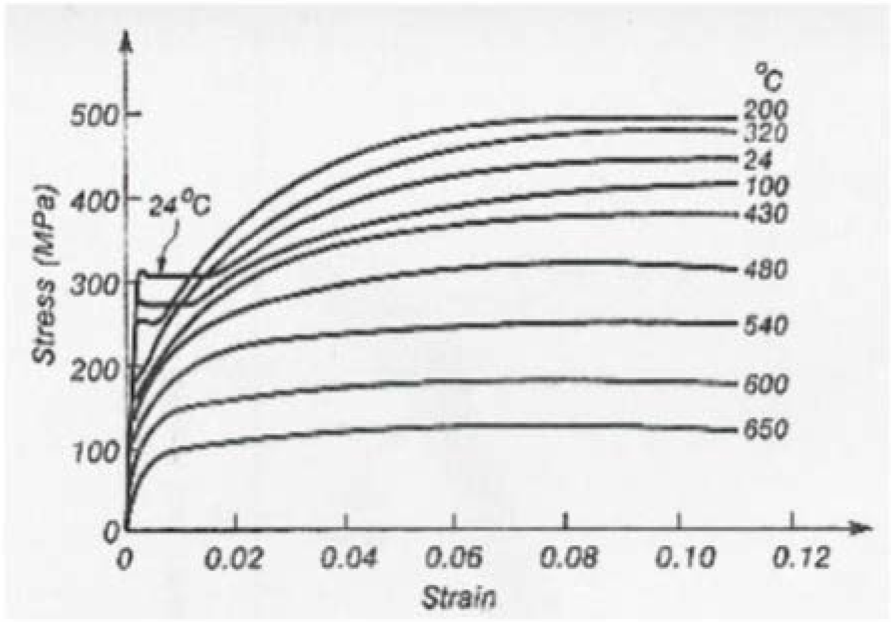


FIGURE 26: STRESS-STRAIN CURVE FOR TYPICAL HOT ROLLED STEEL AT ELEVATED TEMPERATURES[107]

Steel Creep Strain

Welsh [109] has defined creep strain as the permanent deformation of a material that is under constant load. It is also noted that creep is important at elevated temperatures because it can accelerate as load capacity decreases, leading to secondary and tertiary creep, and causing further possibility of plastification and failure. Wright et al. [101] has also explained the importance of creep when steel is exposed to high temperatures, stating that creep becomes more evident when temperatures exceed 400°C. It is also described that creep can be liable for a significant percentage of the permanent deflection observed in steel bridge structures after fire. Wright et al. [101] has also stated that a steel bridge heated and held at a constant temperature can result in slow rise of the structures deflection and possible collapse if duration of the event is long. The effect of creep is explained to be more major for events with longer duration. Buchanan [107] has demonstrated that the creep strain is highly reliant on the temperature and the stress level (as it can be seen from Figure 27 below). This literature has concluded that as temperature rises the creep deformations in steel also rise which can accelerate fast leading to plastic behaviour. Kodur and Dwaikat[105] also concluded in their literature that the degree of creep deformations is influenced by the magnitude and rate of development of temperature and stress in steel. To sum up, literatures reviewed have common conclusion, that increase in temperature influences (increasing) the creep deformation of the steel bridge.

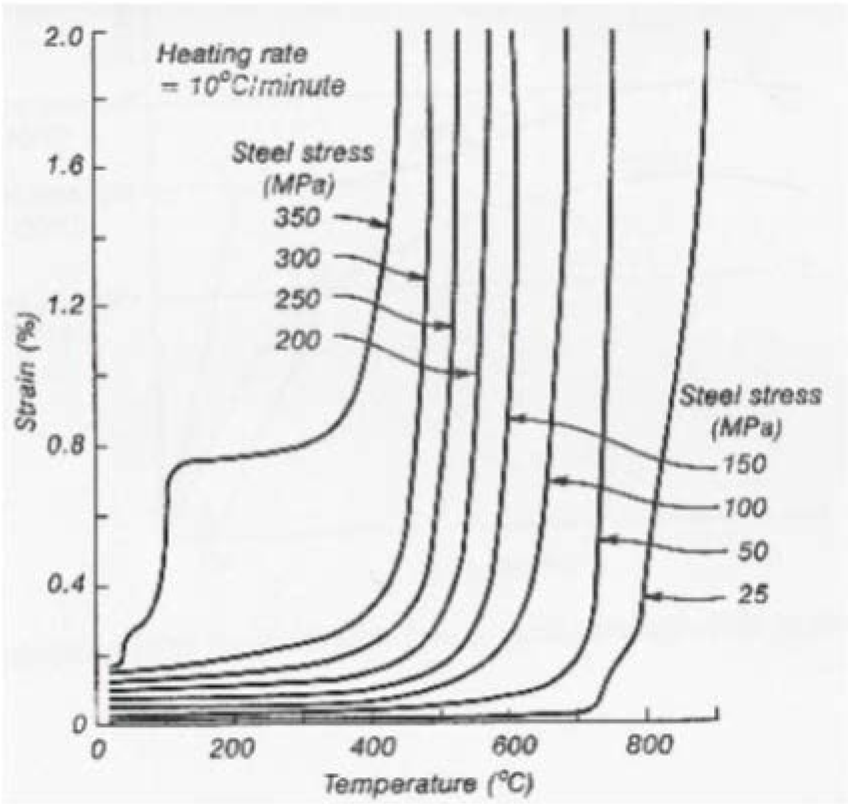


FIGURE 27: CREEP OF STEEL IN TENSION [107]

Concrete Compressive Strength

Wright et al.[101] have explained a main factor affecting compressive strength to be variations in the composition of concrete. It has been described that there is gradual decrease in compressive strength of concrete up to 400°C and steeper reduction at higher temperatures. Astaneh-Asl et al. [103] have explained the compressive strength of concrete to decrease rapidly with the elevation of temperature. The loss of compressive strength of normal weight concrete and light weight concrete due to rise in temperature has been looked at closely with the conclusion of the drop in compressive strength of light weight concrete happening at higher temperatures. Normal strength concrete (NSC) and high strength concrete (HSC) have also been compared inrelation to the decrease in their compressive strength at elevated temperatures (shown inFigure 28) where it has been stated that at high temperatures the strength of high strength concrete decreases to a greater extent than the strength of normal strength concrete [103]. Kodur[110] states that the initial curing, moisture content of concrete, addition of admixtures and binders (such as silica fume) in the concrete mix to be factors that directly impact compressive strength of concrete. This paperinvestigates the effect of high temperatures on the compressive strength of NSC and HSC, reporting a varying rate of decrease in the compressive strength of the two types of concretes due to increase in temperatures. To sum up, all literature report loss of compressive strength of concrete when temperatures increase.

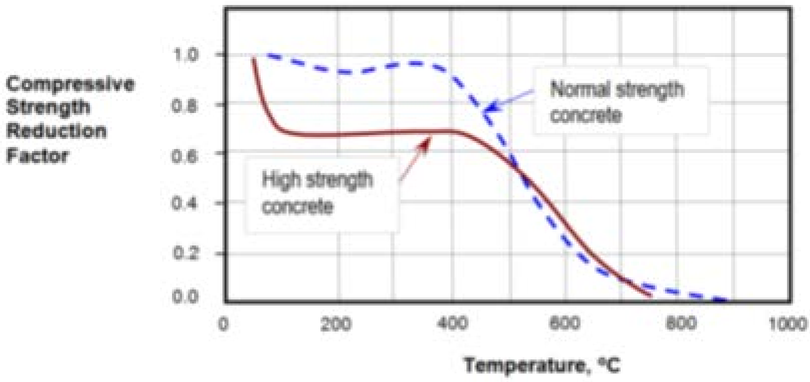


FIGURE 28: REDUCTION IN STRENGTH OF NORMAL STRENGTH AND HIGH STRENGTH CONCRETE IN HIGH TEMPERATURES [103]

Concrete Modulus of Elasticity

Kodur[110] has stated the factors affecting the elastic modulus of concrete to be the water-cement ratio in the mixture, age of the concrete, method of conditioning, the amount and type of aggregates used. He reported that the elastic modulus of both NSC and HSC decreases as temperature increases, this decrease has been associated to the extreme thermal stresses and physical and chemical changes in the microstructure of the concrete. Wright et al. [101] has stated the measurement of elastic modulus to be highly influenced by testing procedures and the type of procedure used. This report concludes that the elastic modulus of concrete decreases with increase in temperature, which it illustrated in Figure 29. Hence the elevation of temperature results in decrease of elastic modulus of concrete.

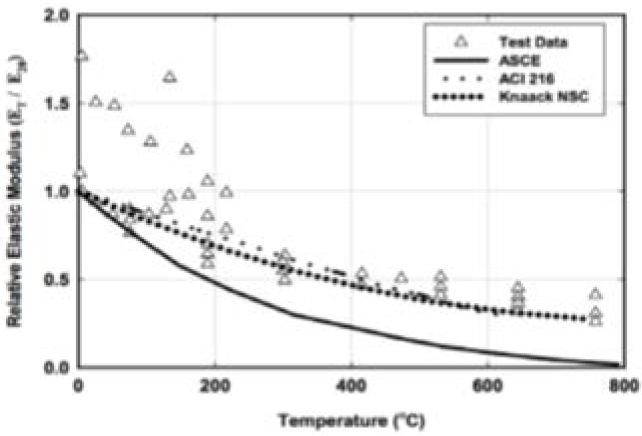


FIGURE 29: REDUCTION IN ELASTIC MODULUS AS TEMPERATURE RISES [101]

Concrete Creep Strain

Kodur[110] has defined creep strain of concrete as the deformation which is dependent on time (duration of the load- such as fire causing increase in temperature). He has concluded that the rise in temperature of concrete under compressive stresses causes increase in creep strain. The increase of creep in concrete at high temperatures is explained to be accelerated by the movement of moisture and dehydration of concrete because of high temperatures and also the increase in the rate of breakage of bond in the cement gel (C-S-H).

Spalling

Another negative effect of heating on concrete is the tendency of the concrete to spall. This is described in literature [111, 112], as pieces of concrete at the surface breaking off. Spalling is described to occur via a number of different ways, sometimes violently with an associated loud bang or crack other times non-violently. Khoury[113]describes great detail when describing the different forms of spalling, identifying explosive/ surface spalling as the most serious form of spalling. This is because spalling at the surface can expose reinforcement bars to fire and/or direct heating, cause significant loss of section, lead to heating at a higher depth of the concrete member and change the parameters used for designing against fire/ heating.

Literature studied asserts that although the absolute intricacies of spalling are not yet fully understood, it is accepted that spalling is most likely a combination of pore water pressure and thermal gradients acting together [113]. Khoury describes a situation where pore water pressure is creating internal stresses laterally, while loading and thermal stresses are acting longitudinally and as a net effect a wedge or trapezoid shaped spall of concrete is forced outwards. Pore pressure is once again a result of the heating and vaporisation of water in the pores of the concrete. There is an allusion to evidence that Ultra High Performance Concrete (UHPC) is more prone to spalling because its structure contains less pores and therefore high pressure water vapour is unable to escape as easily as it may have in normal strength concrete Chan et al. (2000). Concrete may be described as sealed or unsealed on the basis of whether or not it is easy for high pressure water vapour to escape i.e. permeable. Both Khoury[113] and Fletcher et al. [114] describe permeability in this way.

A situation where high thermal gradients caused by rapid heating causing high thermal stresses being combined with a loading which creates high compressive stresses at the surface of the concrete causing spalling is also described. This situation is termed thermal stress spalling.

Concrete bridge failure

Khoury[113] lists 5 modes of failure by which a concrete element may fail as a result of excessive heating due to fire. The first of these is bending-tensile failure as a result of steel reinforcement being heated to a point where its tensile strength is reduced. This mode of failure could be attributed to spalling exposing steel reinforcement bars to heating or at least to a loss of cover. On this topic Fletcher et al. [114] states it is commonly accepted that steel should not be exposed to temperatures above 250°c to 300°c, while steel heated to 700°c steel strength can be reduced to as low as 20% of design strength. Horizontal beams appear to be the most likely to fail in this way.

Other failure methods listed by Khoury[113] are shear-torsion failure, compressive failure and spalling failure. Shear-torsion failure appears to be most likely to occur where shear beam steel reinforcement is compromised. Compressive failure is described to occur in the compression zone of a concrete member under compression such as a column. The failure is a result of heating reducing the compressive strength of the concrete. Spalling failure meanwhile can cause failure through loss of section.

Therefore, it appears as though the failure method of a concrete structure is dependent upon the loading of the specific structural elements which are exposed to fire, the properties of these structural elements such as dimensions and concrete mix properties, the intensity/ duration of the fire, whether reinforcement strength is diminished and whether spalling occurs. Overall, this presents a highly complex system where factors can influence each other and thus the outcome.

Steel bridge failure

A review of the existing literature on failure mechanisms of steel bridges due to fire has revealed that local failure of the bridge girders and deck, shear failure of beams due to high concentrated point loads, the axial restraint conditions on steel beams within the structure and the degree of fire protection present on the bridge during the fire exposure are key areas of interest related to bushfire exposed steel bridges. Each of these above factors has demonstrated a capacity to affect not only the susceptibility to failure during a fire event, but also the residual capacity following exposure.

Mid Span Deflections

As the primary support for steel bridge spans, steel girder failure is synonymous with global failure of a bridge structure. Therefore analysis of this element is essential in establishing a structural vulnerability curve, as well as identifying the ultimate thermal load conditions of the span. When establishing the vulnerability of a structure to bushfire exposure, an imperative criterion for consideration is the likelihood of global survival across a range of fire intensities. The research by Aziz &Kodur[115]analyses an important component of this in their simulation of fire conditions for steel girders, which essentially underpins the global integrity of bridges featuring exposed spans.

Web Deformations

In [116] Glassman andGarlock investigate web shear buckling and the consequences of such deformation for the structural integrity of the bridge. Having identified previous case studies of web shear bucking in fire exposed steel plate girder bridges, Glassman &Garlock utilised Finite Element Modelling to determine the effect of varying Span-to Depth ratios (a/D) across an increasing range of temperatures. The study shows that the ultimate shear buckling strength of web plates post fire-exposure remained relatively high despite exposure to temperatures up to 600 °C. Only the web plate of span/depth ration 3.0 experienced a severe decrease in capacity, retaining only 80% of its original shear buckling strength capacity when it was reloaded post-fire.

Shear Failure

Whilst moment capacity is commonly accepted as the primary mode of failure for fire exposed beams, Kodur and M. Naser's[117]investigate the potential for shear failure under less common loading conditions. Considering of high-shear scenarios such as highly concentrated point loads, beams with slender webs and fire insulated members, several models were developed in finite element modelling softwareconsisting of both structural and thermal elements to investigate fire-induced shear failure modes.

Restrained Beams

Whilst the above failure modes have investigated the structural integrity of primarily steel bridge girders, consideration as to the thermal response of steel beams within a bridge structure should be given. Guo-Qiang and Shi-Xiong[118] and Liu et al. [119] have explored effects of elevated temperature on the restraining beams. The observed deflections and temperature measurements of both specimens provided an excellent insight into the transformation of a restrained beam during fire exposure, specifically with regard to the compressive and tensile stresses experienced within the member. The transition from compression (restrained thermal expansion) to Tension (Cantenary action succeeds thermal expansion) followed again by tension (contraction due to cooling) reveals the likely structural forces imposed by a restrained member during fire exposure, which may be considered when analysing a fire affected steel bridge structure.

Fire Protection Measures

In the protection of steel structures against fire-related damage, a general three component approach namely: active, passive and preventative measures are taken to minimise the potential and consequence of fire with respect to structural integrity. Whilst active and preventative strategies are limited in the context of steel bridge design, many viable opportunities for passive protection exist.

Passive protection methods occasionally applied where cost benefits outweigh the potential performance benefits provided by comprehensive protection. As discussed in [120], partial fire protection may be applied on composite beams without sacrificing the structural integrity of the member. The significance of this lies in the cost savings realised through simplified application, a design consideration relevant to all infrastructure projects. Alternative methods of partial protection such as web and block infilled columns, slim floor beams and full/partial concrete encasement of steel members may also be worth contemplating in the context of steel bridge vulnerability.

CASE STUDIES

In order to understand the failure of bridge structures under natural hazards, two case study analyses were undertaken. The case studies assisted in linking the intensity of hazards to the failure modes and mechanisms.

The preliminary analysis presented here will be further improved with subsequent collection of data on natural hazards which will lead to quantification of natural hazard induced loading on bridge structures.

The analysis also explores development of vulnerability models which can contribute towards quantifying the impact of hazards on bridge structures.

CASE STUDY1

Introduction

Australia has been prone to bush fires with 136 towns reportedly affected between years 1851-2009. Direct impacts of bush fires include damaged assets as well as casualties during the bushfire events whilst indirect impacts include service disruptions, loss of income and trauma. BITRE estimated 8.2 million dollars as an average annual cost of bushfires in Australia between 1967 and 2005 from which the state of Victoria has the highest proportion of 37% among other states (BITRE, 2008).

Case study 1, presents an analysis of a selected concrete bridge in Victoria when exposed to bush fires.

The behaviour of reinforced concrete under extreme temperature has been modeled and studied by numerous researchers (Terro, 1998, Khoury, 2000, Dotreppe et al., 1997. VicRoads, the state of Victoria's road authority has published a technical note on fire damage in reinforced concrete and with recommendation regarding assessment and repair practice on the affected components (Andrews-Phaedons, 2011). Required repair works for concrete under fire have been recommended in Lin et al., (1995), Garlock et al., (2012), andYaqub and Bailey, 2011. Furthermore, risk evaluation and damage indices have been investigated by Blong and Blanchi (Blong, 2003a, Blong, 2003b, Blanchi et al., 2002). However, a systematic method of assessing bridges prior to a bush fire event to establish the probability of failure is a current gap in knowledge. This paper presents a simplified method for assessing reinforced concrete bridges considering three possible failure scenarios.

Methods and recommendations for assessment of concrete bridge structures under the extreme temperature of fire were reviewed prior to commencing the analysis. Isotherm method is used for assessment of the bridge members where time and temperature are the variables in the study. Risk of failure has been evaluated and repair strategies have been recommended.The isotherm methodology is applied to ascertain thebridge structural behaviour not only to identify potential damage and recommend repair work, but also to evaluate the risk and damage index in concrete bridges under extreme heat based on fire exposure duration. The case study is presented to demonstrate the methodology of assessment of a bridge structure. Presented process will assist road authorities to predict the potential damage to the road bridges and to proactively initiate strengthening programs to prevent catastrophic events or to prepare for alternative strategies at the time of disasters. Furthermore, emergency services can be informed of the potential damages and risks of using the road network in the response time at a bushfire event. In addition, cost estimations can be made for recovery of the damaged bridges using the recommended repair works. Therefore, the paper creates a seamless procedure for emergency management of concrete bridges to cover the stages of Prevention, Preparedness, Response and Recovery (PPRR).

Review of bushfire impacts on concrete structures and methodologies

Literature and standards have been published to address the need for designing structures under the extreme heat of fire. There are a number of descriptive codes which cover deem to satisfy design of elements in extreme heat, which provide tabulated recommendations for members' dimensions and minimum covers for standard fire endurance. However, European codes have pioneered the use of performance based design methodologies. The second chapter of the ACI/TMS 216 and also the section 4 of the BS 8110 Part 2 specify requirements for determining fire resistance of concrete elements based on dimensions and minimum cover (ACI 2007, BS 1985). However, the British standard has been replaced by the Eurocode 2 since 2010. Structural components' fire testing methods are described in standards such as AS 1530.4 (2014), BS 476 and ASTM E119 (2014) in which testing procedures for construction materials are provided. Furthermore, national building codes provide specific requirements for fire resistance in buildings construction and selection of materials. National Building Code of Canada (NBC 2010), National Fire Code of Canada (NFC 2010) and the Building code of Australia (BCA) (ABCB 2014) are examples of these codes .

Eurocode 2 (EN 2004) covers fire design for concrete structures. The code provides 3 different methods 1. tabulated data, 2. simplified calculation methods and 3. advanced calculation methods for designing concrete elements. Use of the tabulated data is simple; however, it has restrictions such as up to 240 minutes of fire exposure could only be considered using this method. Simplified methods which consist of 500ºC isotherm method (reduced section method) and the zone method (method of slices) can be used for standard and parametric fire events (EN 2004, Purkiss 2007). However, for global structural analysis, advanced calculation models are recommended by the Eurocode 2 (EN 2004). Phan et al. (Phan et al. 2010) states the BS 7974 as the most comprehensive code of practice for specific fire engineering design in any country. The code provides complementary guidance to Eurocode for calculation of structural fire resistance.

Overall impact

The impact of the elevated temperature caused by fire on material types used in construction of bridges could lead to degradation of structural or functional capacity of the structures and eventually failure of their elements. Responses of structures exposed to fire can vary, however, they could be categorised in thermal, mechanical & deformation responses. Some of the thermal properties of concrete affected by increase in temperature are thermal conductivity, specific heat, and thermal elongation (Li et al. 2003). Some of the mechanical properties of concrete affected by increase in temperature are the compressive strength, tensile strength, elastic modulus and creep strain.

Kodur (2014) states that the response of concrete to elevated temperatures are affected by temperature changes, composition, characteristics of concrete batch mix, heating rate and environmental conditions. Li et al. (2003) state that concrete is a composite material meaning the components will have different thermal characteristics and that concrete has properties which depend on moisture and porosity. Bilow and Kamara (2008) state that changes in properties of concrete at elevated temperatures are influenced by the type of coarse aggregate used in the concrete, the coarse aggregate being classified into three types: carbonate, siliceous and lightweight. In concrete, the high temperature of fire causes self-destructive stresses as well as chemical reactions, which create cracks, spalling and weakening of strength, stiffness and ductility of the concrete as a material (Astaneh-Asl et al. 2009). According to Phan et al. (2010), fire design would be the same as a normal structural design if the designer considers the following 7 points:

- Load changes on the structure during the fire
- Internal forces due to thermal expansion
- Strength reduction of the materials
- Cross section reduction of structural elements
- Reduction of safety factors due to smaller likelihood of the consequence
- Structural members deflection consideration
- Consideration of all possible failure mechanism

Typical failure modes of concrete bridges during a bushfire

Although concrete is one of the most resistance materials among the conventional bridge construction material, being exposed to extreme heat of fire, local and eventually global failure are inevitable in extreme cases. Common local failure mechanisms of concrete members under extreme heat are:

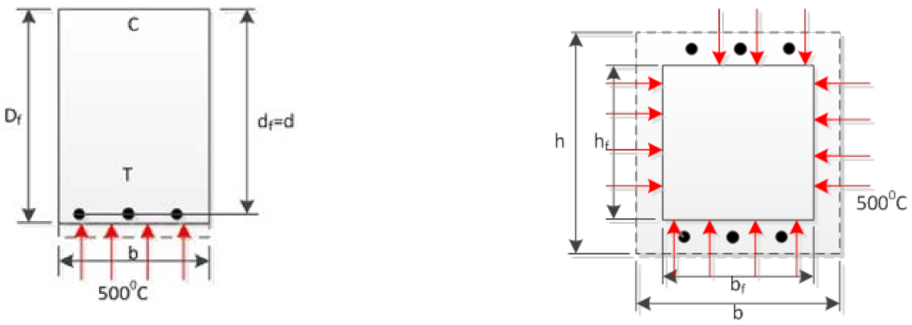
- Concrete spalling
- Concrete cracking
- Concrete delamination
- Compressive strength reduction
- Steel reinforcement and prestressed strands strength reduction

Methodology

500^{0}C isotherm method described in Eurocode for a standard fire exposure is used in this analysis. Reduced cross section is calculated at the beginning and then the reduction in the steel strength is calculated based on the data given in Eurocode 2. Afterwards, traditional calculation method can be adopted to find the moment capacity of the reduced section.

Reduced cross section at elevated temperature

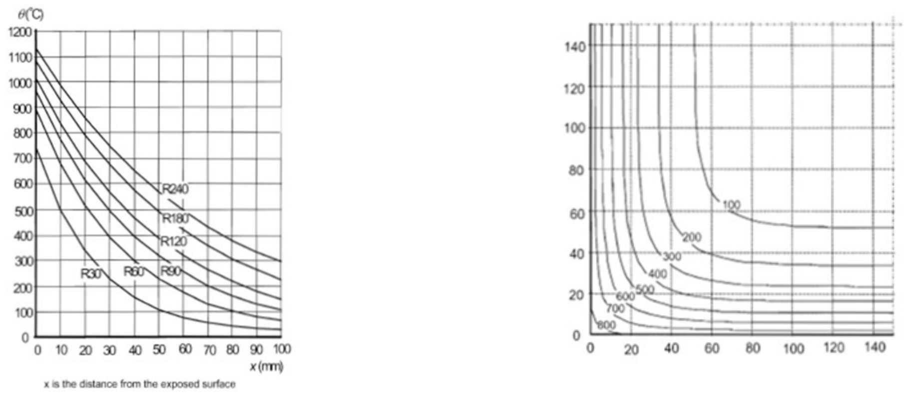
Damaged concrete is assumed not to contribute to the load bearing capacity of the member(Eurocode 2).Heat damaged zone (i.e. concrete with temperatures in excess of 500°C) at the concrete surface is disregarded and a reduced cross section thus resulted in is considered in the analysis. Figure 21(a) shows the reduced cross section of reinforced concrete slab fire exposure on one side while Figure 21(b) shows the same for a column with fire exposure on all four sides. The residual concrete cross-section retains its initial values of strength and modulus of elasticity.



(a). Slab (b). Column

FIGURE 30:REDUCED CROSS SECTION OF REINFORCED CONCRETE MEMBERS

In order to find the isotherm of 500°C for different exposure times for slab and column the figures given in Eurocode 2 were used (Figure 2). Figure 2(b) shows the temperature profiles only for an exposure time of 30 minutes as an example of all the others available in the Eurocode for more exposure classifications. Position of T500 for columns was calculated using the average of the minimum (from the edge) and the maximum (from the corner).



(a). Slab (b). Column (R30)

FIGURE 31: TEMPERATURE PROFILES

Strength of steel at elevated temperature

Distance to the center of the reinforcing bars needs to be figured out using the cover. The temperature of the individual bars (taken to the center of the bar) can be obtained using Figure 2. Although some of the reinforcing bars may fall outside the new reduced cross section as shown in Figure 1(b), they will be included in calculating the ultimate moment capacity provided that the tensile strength is adequate. Strength reduction factors are given for tension and compression reinforcement for Class N and Class X types in the form of tables and equations in Eurocode 2. Due to the limitations in the length of the paper, those tables and equations have been omitted.

Failure Conditions

There are three scenarios where potential damage to the bridge and its strength should be considered.

1. The first scenario is during fire under dead load, where the strength of the members drop to such a degree that the structure can no longer support its self-weight. This is a critical failure condition, as no amount of emergency response (such as cutting off traffic) or remedial work can be undertaken to reduce the damage.

   Failure can be said to occur when the temperature in the rebar reaches 593°C which corresponds to 50% loss of steel strength. (Raut&Kodur 2009)

   The damage for this situation will be assessed using the reduced yield strength of reinforcing at the max temperature reached, and the reduced strength of concrete at max temperature reached, where all areas of concrete that have reached 500°C are counted as having f'c = 0.

2. Fire under dead and live load, where a vehicle will attempt to use the bridge during the fire event will be the second scenario. This will not be counted as a critical failure condition as it likely that traffic will not attempt to cross the bridge during the fire, and if it does so complete failure is much more likely making modelling of the degree of damage pointless.

3. The third scenario is after fire under dead and live load, where the residual strength of the members (after the steel strength has recovered to normal temperatures) is still not sufficient to support traffic loading.

   The damage for this situation will be assessed using the residual reduced yield strength of reinforcing at the max temperature reached, and the reduced strength of concrete at the max temperature reached, where all areas of concrete that have reached 500°C are counted as having f'c = 0.

   It is assumed that where any change in strength of the bridge is observed post-fire, repair will be required to return the bridge to pre fire capacity.

Studied Bridge

While an extensive amount of bridges are in use in Victoria,an older structure will be used in this cases study assessment with the age ranging from 50 – 59 years. Both reinforced flat slab bridges and reinforced decking unit bridges common through the region will be assessed. Based on the standards of the time an assumed cover depth of 30mm in beams/slabs and 40mm in columns will be used.

FIGURE 32: CASE STUDY BRIDGE

The bridge was constructed in 1958 and consists of reinforced concrete columns, diaphragms and 500mm deep deck slab (Figure 23). The structure comprises six piers and concrete abutments. Piers 1 and 6 comprise 5 columns each and have pinned connections to the deck and piers 2 – 5 have 6 columns each and are cast integrally with the deck.

The waterway being crossed is a wide stream which fluctuates at different times of the year. This waterway has abundant vegetation, weed and some debris which may hinder the flow.

The columns, crossheads and abutments appear to be generally in good condition although typical hairline to medium transverse and longitudinal cracking has developed in several locations. Abrasion of the concrete due to water wash was evident at the base of all columns.

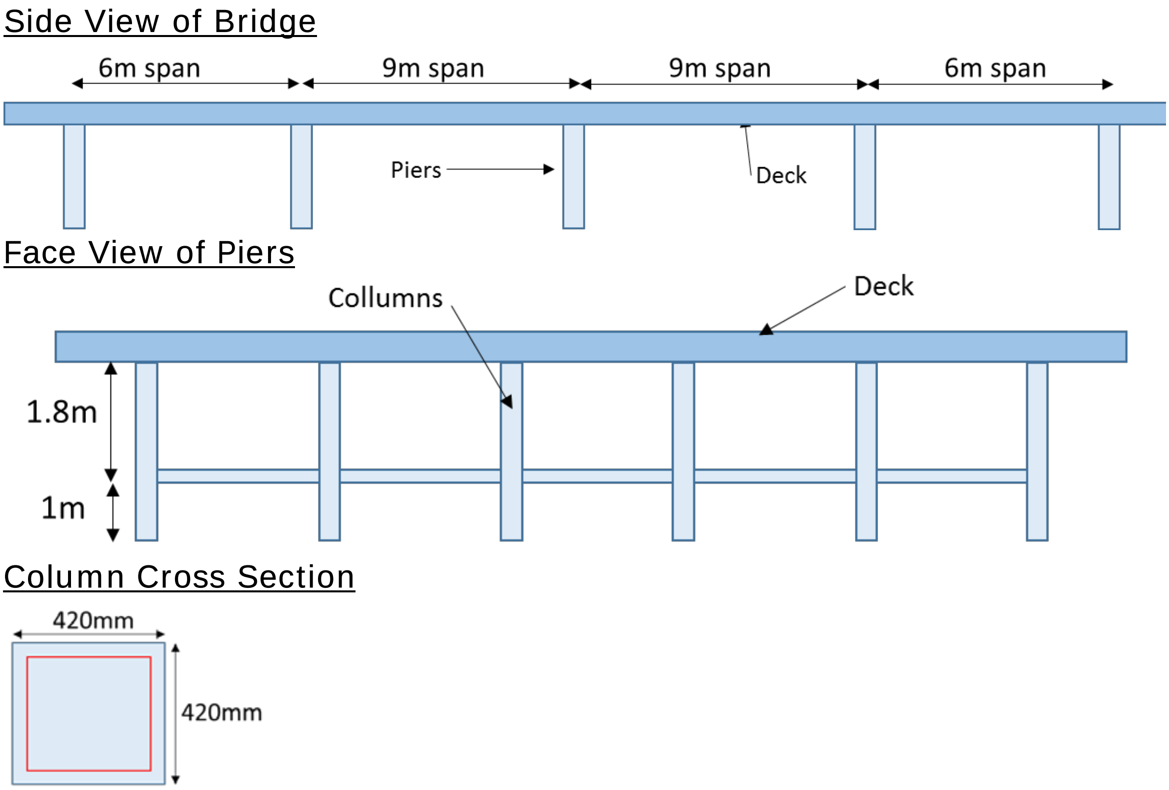


FIGURE 33: DIMENSIONS OF THE BRIDGE

Analysis

Deck slabs and deck units

Table 10shows the depth of the isotherm 500 as well as the concrete reduction coefficient based on the fire exposure time. The temperature in the reinforcement bars and their corresponding yield strength and residual yield strength reduction factors are given in Table 11for concrete deck slabs.

TABLE 10:DEPTH OF 500°C ISOTHERM, AND CONCRETE REDUCTION COEFFICIENT, KC VALUES.

Depth of T500		K_c (at depth from exposed surface)			
time	mm	50mm	100mm	150mm	200mm
30	10	0.88	1	1	1
60	21	0.64	0.975	1	1
90	29	0.43	0.92	1	1
120	36	0.3	0.825	0.99	1
180	49	0.15	0.64	0.95	1

TABLE 11: TEMPERATURE OF REINFORCEMENT AND ASSOCIATED YIELD STRENGTH REDUCTION FACTOR(R), AND RESIDUAL YIELD STRENGTH REDUCTION FACTOR (RRESIDUAL).

Temperature at 30mm (reinforcement)			
time	T(°C)	r	$r_{residual}$
30	230	1	1
60	395	0.649	1
90	495	0.436	1
120	570	0.277	0.93
180	680	0.043	0.82

Table 12shows the bending strength and the stiffness reduction factors for during and after the extreme heat on the concrete slab.

TABLE 12: : REDUCTION FACTORS FOR BENDING STRENGTH (MU FACTOR) AND MEMBER STIFFNESS (STIFFNESS FACTOR) OF SN5577'S REINFORCED CONCRETE DECK SLAB.

		Mid span			Above Pier			
			Mu factor			Mu factor		
	B (mm)	d(mm)	During Fire	After Fire	d(mm)	During and After Fire	$K_{c,mean}$	stiffness factor
T(30)	610	270	1.000	1.000	260	0.963	0.951	0.803
T(60)	599	270	0.650	1.000	249	0.922	0.926	0.667
T(90)	591	270	0.438	1.000	241	0.892	0.910	0.581
T(120)	584	270	0.278	0.930	234	0.866	0.897	0.516

T(180)	571	270	0.043	0.821	221	0.818	0.884	0.422

Columns

The depth of the Isotherm 500 and the corresponding concrete strength reduction coefficient in columns are given in Table 13. Table 14illustrates bending strength reduction factors, compression capacity reduction factor and the stiffness reduction factor for during and after the fire exposure on columns.

TABLE 13: DEPTH OF 500°C ISOTHERM AT AND CONCRETE REDUCTION COEFFICIENT, KC VALUES.

Position of T500				kc			
time	Minimum (mm)	Maximum (mm)	Average (mm)	50mm	100mm	150mm	200mm
30	10	22	16	0.88	1	1	1
60	22	39	30.5	0.64	0.975	1	1
90	32	50	41	0.43	0.92	1	1
120	40	61	50.5	0.3	0.825	0.99	1
180	50	70	60	0.15	0.64	0.95	1

TABLE 14: REDUCTION FACTORS FOR MEMBER BENDING STRENGTH (MU FACTOR), MEMBER COMPRESSION CAPACITY (N FACTOR) AND MEMBER STIFFNESS (STIFFNESS FACTOR), AS WELL AS EFFECTIVE LENGTH AND RADIUS OF GYRATION RATIO OF SN5577'S REINFORCED CONCRETE COLUMNS.

				Mu factor				N Factor			
	B(mm)	D(mm)	d(mm)	During Fire	After Fire	radius of gyration	Le/r	During Fire	After Fire	k	stiffness factor
T(0)	420	420	380	1	1	121.8	14.8	1	1	1	1
T(30)	388	388	364	0.957	0.957	112.5	16.0	0.881	0.881	0.898	0.587
T(60)	359	359	349.5	0.721	0.918	104.1	17.3	0.738	0.782	0.87	0.404
T(90)	338	338	339	0.476	0.889	98.0	18.4	0.620	0.715	0.86	0.310
T(120)	319	319	329.5	0.292	0.827	92.5	19.5	0.523	0.649	0.855	0.243

Results

The following table (Table 15) shows the estimated damages to the deck and columns of the case study bridge in fire exposure durations of 30, 60, 90 and 120 minutes. Rehabilitation or replacement actions are also suggested based on the estimated damage on the components.

TABLE 15: DAMAGE AND REPAIR REQUIREMENTS

Exposure Time	Deck Units	Columns
30 minutes	500°C isotherm 10mm deep +	500°C isotherm 16mm deep +

	cracking. Post fire yield strength of reinforcement is unaffected. Repairing of damaged concrete required.	cracking. Post fire yield strength of reinforcement is unaffected. Repairing of damaged concrete required.
60 minutes	500°C isotherm 21mm deep + cracking. Post fire yield strength of reinforcement is unaffected. Repairing of damaged concrete required.	500°C isotherm 30.5mm deep + cracking. Post fire yield strength of reinforcement is unaffected. Repairing of damaged concrete required.
90 minutes	Ruined concrete (500°C Isotherm) has reached reinforcement. (30mm) Post fire yield strength of reinforcement is unaffected. Repairing of damaged concrete required.	500°C Isotherm is average of 10.5mm past reinforcement. (40mm) Post fire yield strength of reinforcement is unaffected. Repairing of damaged concrete required.
120 minutes	500°C Isotherm is 6mm past reinforcement. Post fire yield strength of reinforcement is reduced by 7% Repairing of damaged concrete required.	500C Isotherm is average of 20mm past reinforcement. Post fire yield strength of reinforcement is reduced by 4%. Replacement of the columns required.

Risk of failure of Bridges

Based on the structural capacity reductions calculated in Section 5, failure risks of the components have been suggested inTable 16.

TABLE 16: RELEVANT VALUES FOR FAILURE CONDITION 1: DURING FIRE UNDER DEAD LOAD

Exposure Time	Deck Units	Columns
30 minutes	Stiffness has dropped by close to 11%.	Moment capacity has dropped by 4%, compression capacity has dropped by 12%, and stiffness has

	No risk of failure.	dropped by 41%. No risk of failure.
60 minutes	Sagging moment capacity has dropped by 35%, and stiffness by 20%. Failure unlikely since the bridge will only be supporting the deadload. Small amount of extra damage from deflection likely.	Moment capacity has dropped by 28%, compression capacity has dropped by 26%, and stiffness has dropped by 60%. Failure unlikely since the bridge will only be supporting the deadload.
90 minutes	Sagging moment capacity has dropped by 56%, and stiffness by 25%. Failure unlikely. Extra damage from deflection likely.	Moment capacity has dropped by 52%, compression capacity has dropped by 38%, and stiffness has dropped by 69%. Buckling Failure possible.
120 minutes	Sagging moment capacity has dropped by 72%, and stiffness by 29%. Flexural Failure possible. Extra damage from deflection likely.	Moment capacity has dropped by 71%, compression capacity has dropped by 48%, and stiffness has dropped by 76%. Buckling or compression Failure possible.

Conclusions

The case studyexplored extreme fire impacts on concrete bridges and presented a methodology to estimate the extent of damages on concrete structures. Isotherm 500 method has been utilized to analyze a case study bridge in Victoria due to effects of extreme heat. The extent of fire damage and resulting strength reduction in the bridge deck and columns have been investigated during and after the fire. Rehabilitation or replacement actions as well as failure probability estimations have been presented. Following conclusions can be made from the outcome of the analysis of the case study:

- Columns were significnatly at a higher risk than the slab due to their exposure to fire on all sides. Also, the duration of exposure would be higher for the columns as well in a real situation. If the exposure was limited to 90 minutes, the bridge could be repaired to its pre-disaster capacity

- If the duration of exposure is over 120 minutes, all the columns of the bridge would require full replacement. The columns have a high risk of failure under fire as well, which may lead to a need for full replacement of thebridge.

Whilst the analysis was limited to one bridge, the generic process can be adopted for other bridges of the network to ascertain the risk of damage under Bush Fire. Critical bridges in high risk regions can be hardened to ensure that failure doesn't occur under common exposure scenarios.

CASE STUDY 2

Failure analysis of typical bridge type due to earthquake

This section provides information on failure mechanism of a typical bridge type in Australia and a probability based fragility curve methodology was developed to find the vulnerability of these types of bridges.

Introduction

It is well known that significant uncertainties are involved in the estimation of ground motion, seismic demand, and seismic capacity of a bridge. To incorporate these uncertainties in the bridge vulnerability assessment, a probabilistic seismic damage analysis or fragility analysis of bridges is performed and the results are expressed in fragility curves. A fragility curve displays the probability that a bridge is being damaged beyond a specified damage state at various levels of ground shaking.

Methodology

Bridges cannot be independently assessed for vulnerability of road infrastructure, as there is a significant impact on the operation of road network during and after a natural disaster such as an earthquake event. However in this study of vulnerability assessment, only bridge failures are considered independent of the road network, since this proposed methodology will be improved for the entire road infrastructure in future studies. There are a lack of cases in Australia where the cause of bridge failure has been identified. Identifying potential weak points ensures that the capacities and thus vulnerability are correctly gauged. Therefore as the first step of this study, the failure mechanism and critical structural components of a typical girder bridge in Australia is identified based on the level of the damage in each structural component .

Seismic risk cannot be obtained without quantification of the impact of structural components of the bridge. The major risk is associated with the vulnerability of bridges with the effect of specific characteristics of the seismic event. Thus the failure mechanism of the bridge is identified, probabilistic seismic risk assessment is carried out using fragility curve method, as behaviour of bridges due to earthquake excitation is highly variable. Generation of fragility curve methodology is developed using full non-linear time history analyses with particular damage states of critical components in the bridge. The vulnerability of the bridge is expressed in terms of predefined damage states based on capacity and demand ratios of the significant structural components.

Bridge characteristics and Non linear time history analyses

The most common bridge type in Australia is precast concrete girder bridges. The Tenthill creek bridge is selected for the study and characteristics of the bridge is collected from previous studies [121]. The Tenthill creek bridge is a simple span reinforced concrete bridge built in 1976 to carry a state highway in

Gatton, Queensland. The bridge is 82.15m long and about 8.6m wide. It is supported by a total of 12 pre-stressed 27.38m long beams over three spans of 27.38m. Both ends are supported by two abutments and two headstocks as shown in Figure 1.

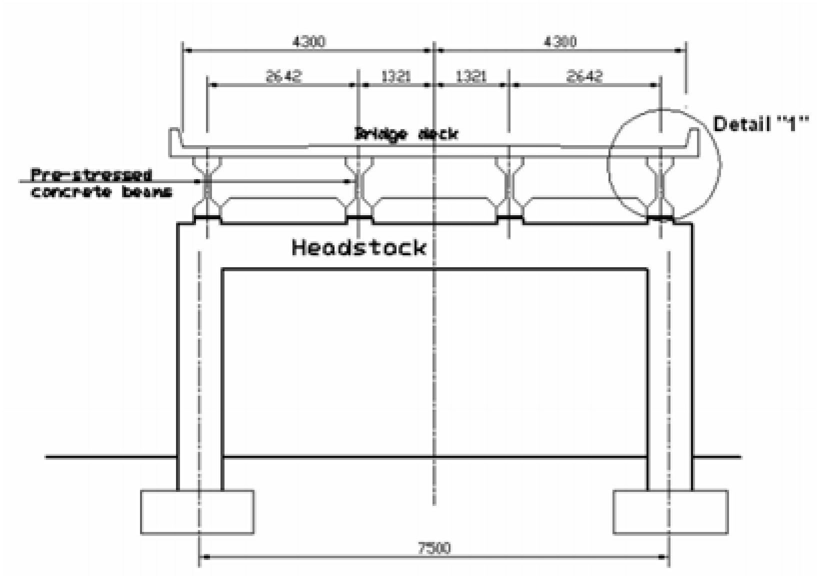


FIGURE 1: SECTION DETAIL OF THE TENTHILL CREEK BRIDGE (SETUNGA ET AL., 2002)

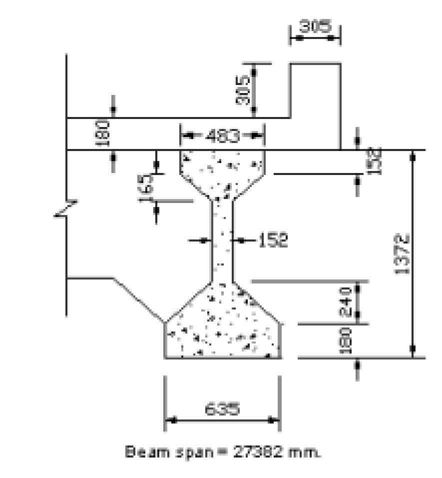


FIGURE 2: SECTION DETAIL OF PRE-CAST CONCRETE BEAM (SETUNGE ET AL., 2002)

Three dimensional analytical models are developed using ANSYS 14.5 software. All the structural components are modelled as per the structural drawings except the abutments at the two end. The boundary conditions for these ends are given as roller supports to allow displacement in horizontal direction. The nominal yield strength of main reinforcement is 400MPa and shear reinforcement is 240MPa. The nominal concrete compressive strength is 20MPa [121].

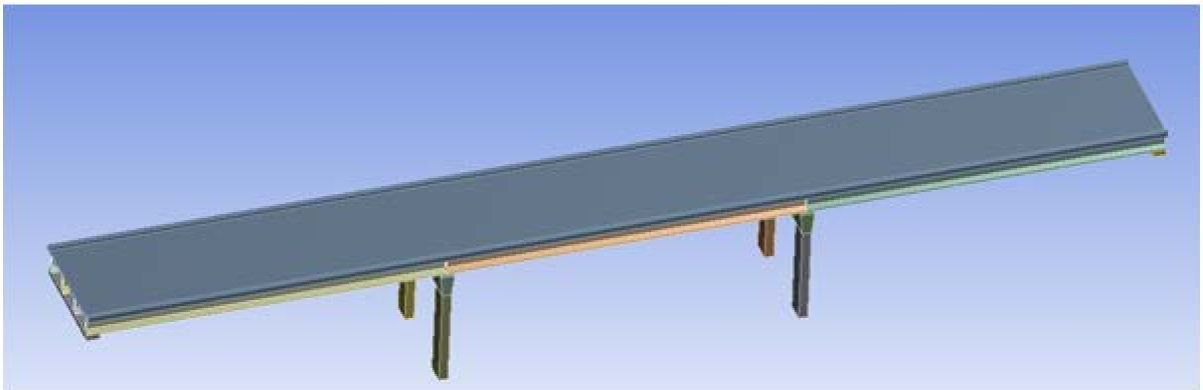


FIGURE : GEOMETRIC MODEL CREATED IN ANSYS (ELEVATION)

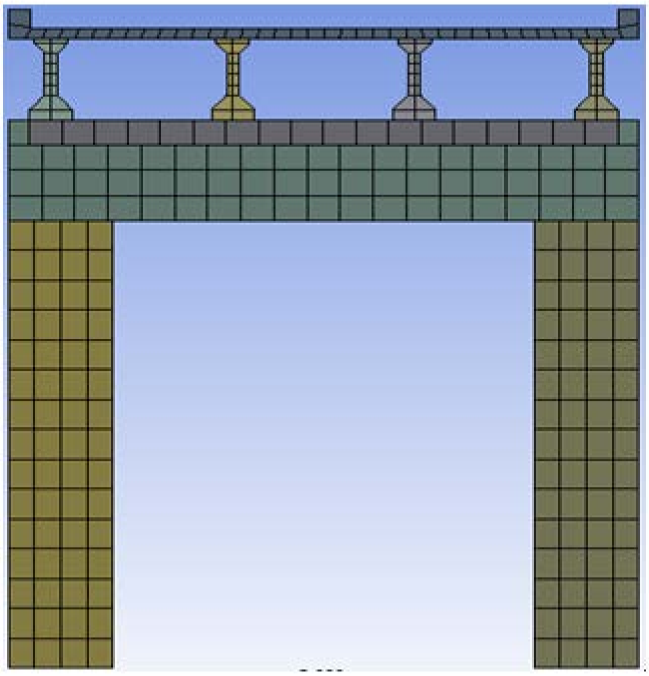


FIGURE 4: GEOMETRIC MODEL CREATED IN ANSYS (SECTION)

Ground motion selection

Since the available past earthquake time history records are not much available for Australia, artificially generated earthquake time histories were used in this study. Rock sites and different magnitudes and focal distances were considered with varying peak ground acceleration ranging from 0.07g to 0.17g. Total number of 24 artificially generated earthquake time histories were used in this study. The PGA and the number of each accelerogram used are shown in Table 1 and a typical accelerogram is shown in Figure 5.

TABLE 1 : EARTHQUAKE TIME HISTORIES USED IN THE STUDY

PGA	Number of accelerograms
	Rock site
0.07	5
0.08	4
0.10	2
0.11	6
0.13	1
0.14	1
0.15	2
0.16	2
0.17	1

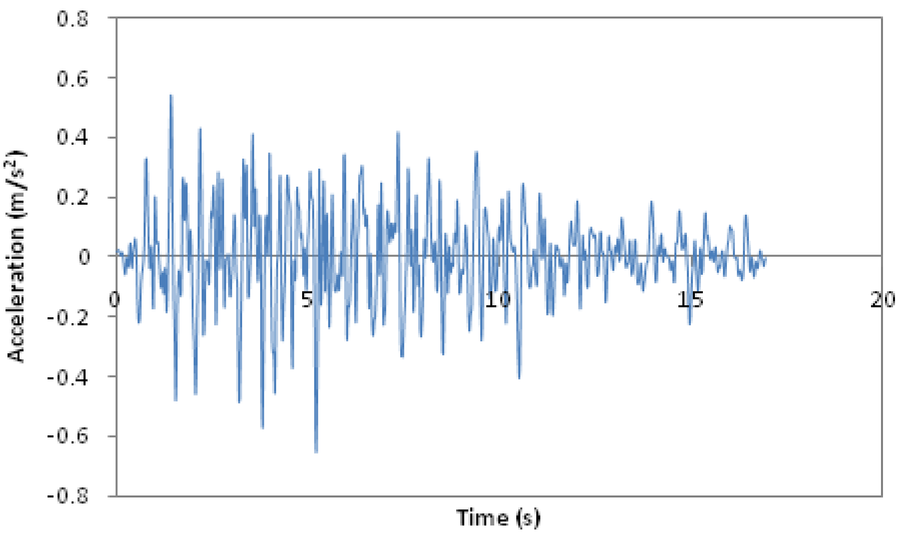


FIGURE 5: TYPICAL EARTHQUAKE TIME HISTORY USED IN THE STUDY

Although there are different types of bridge types available in Australia, it is time consuming to conduct a vulnerability assessment for all the types of bridges. Therefore in this study, a typical prestressed girder bridge was considered to represent the bridge types in Australia. However this study is continuing to develop fragility curves for other types of bridge types.

Damage state definitions

In the PBEE framework of PEER, an engineering demand parameter (EDP) is designated to represent the response of a structure subjected to an earthquake motion and a damage measure (DM) is defined to describe the damage in the structure. In most study cases, a DM is treated as a redundant interim variable as a reasonable EDP is able to depict the damage status of a structure.

In fragility analyses, damage probabilities are usually given with several damage states, characterizing several levels of performance. A meaningful definition of damage states, assisted with accurate limit state which should be provided and validated by visual inspection and/or analytical results, can not only predict structural damage gradually but also well correlate damage states with physical phenomenon. Two sets of well-developed limit state definition systems nowadays are from HAZUS and European Code.

Based on the HAZUS damage states, Basoz and Kiremidjian [122] made a detailed bridge damage criteria description system and used it in the empirical study by Basoz et al. [122]. Dolsek and Fajfar [123] incorporated the regulation about damage in European performance-based design code into the fragility analysis, and three damage states are defined as 'damage limitation (DL)', 'significant damage (SD)' and 'non collapse (NC)'.

Even though both of the above two damage state definition systems present corresponding physical phenomenon, it is still hard for some analysis to find reliable limit state due to the difficulty to relate the description to quantitative EDP, especially when the components other than columns are investigated and the EDP has not been well studied yet. Therefore, in lots of study cases, only the fragility curves of damage states corresponding with first yielding and collapse are plotted. Another way to dodge the difficulty of defining reasonable limit state is to plot fragility curves versus continuous limit state instead of certain limit state with meaningful damage states.

A great amount of studies have been done to define damage index and quantify limit state based on the damage in various components. In contrast to component specified damage measure, one can also define general bridge DM. Jalayer et al. [124] employed the critical demand-to-capacity ratio, which is the ratio of the demand on a component or a damage mechanism and to the capacity corresponding to structure closet or collapse failure:

Damage Index (DI) = D/C

where C denotes the demand and D is the maximum capacity of the investigated component.

Mackie and Stojadinovic [125] used traffic volume loss and loss of longitudinal or vertical load carrying capacity as damage measure. Another appealing global level DI is the expected economic loss. Goulet et al. [126] illustrated the idea of evaluating the damage by repair cost. The expected annual loss of a RC frame building is calculated by the authors. Mackie and Stojadinovic [125] also developed a DI named repair cost ratio (RCR) which is the ratio of repair cost to replacement cost.

Development of Fragility curves

Being a probabilistic methodology developed in the last two decades, fragility theory employs a certain type of expressions to address the uncertainties and demonstrate the reliability of structures achieving certain performance objectives during earthquake. Knowledge and methods from both performance-based earthquake engineering (PBEE) and seismic reliability contributes to the prosperity of the fragility function theories and applications.

Fragility function method is essentially a branch of seismic reliability analysis methodology, expressing the reliability in a certain way. Fragility could be defined as the conditional probability of failure or exceeding a prescribed limit state, given a level of earthquake intensity [124]. Consequently the fragility function describes the probability of failure to meet a performance objective as a function of demand on system [127].

Fragility analysis was developed as a probabilistic methodology which addresses the uncertainties and demonstrates the reliability of structures achieving certain performance level. There are two types of fragility curves namely empirical and analytical. Empirical fragility curves are derived based on damages of bridges due to past earthquakes while analytical fragility curves are developed through seismic response of bridges with analytical results. Many researchers involved in developing some empirical fragility curves for bridges due to past earthquakes [122, 128, 129]. Most bridges in Australia were not designed to resist earthquakes and damage data is not available. Therefore an analytical based approach is used in this study to develop fragility curves for the bridge.

Developing analytical fragility curves can be obtained from non-linear time history analysis, elastic spectral analysis or non-linear static analysis. Hwang et al., [130] proposed an analytical method based on capacity and demand of the bridge components that is potentially being damaged due to seismic event. This method was used in evaluating the seismic damage of the highway systems in Memphis by Hwang et al., [81]. Tavares et al., [131] assessed the seismic vulnerability of typical bridge types in Quebec through fragility curves. It was found from the study that concrete girder bridges are more vulnerable to seismic damage than steel bridges. Further, Choi et al., [132] developed some fragility curves for bridges in Central and Southern United States and observed simply supported concrete girder bridges are more susceptible to seismic damage.

Definitions of Damage States and Corresponding C/D Ratios [81]

Damage state	**Description**	**C/D Ratios**
No Damage	Although minor inelastic response may occur post-earthquake damage is limited to narrow cracking in concrete. Permanent deformations are not apparent	$\frac{C}{D} \geq 0.5$
Repairable damage	Inelastic response may occur, resulting in concrete cracking, reinforcement yield and minor spalling of cover concrete. Extent of damage should	$0.5 > \frac{C}{D} \geq 0.33$

	be sufficiently limited so that the structure can be restored essentially to its pre-earthquake condition without replacement of reinforcement or replacement if structural members. Repair should not require closure. Permanent offsets should be avoided.	
Significant damage	Although there is minimum risk of collapse, permanent offsets may occur, and damage consisting of cracking, reinforcement yielding, and major spalling of concrete may require closure to repair. Partial or complete replacement may be required in some cases.	$\frac{C}{D} < 0.33$

In the proposed approach, several bridge models were established for the bridge. The bridge deck connection that has the potential for being damaged during an earthquake was evaluated to determine their capacity/demand (C/D) ratios.

In this study, seismic fragility analysis of the inventory of Memphis and Shelby County bridges was performed using the C/D ratio method in accordance with the FHWA seismic retrofit manual. The results of the forces or displacement "demands" are calculated from an elastic spectral analysis. The "demands" are compared to the "capacities" of each bridge component to resist these forces or displacements

Results and discussion

Non-linear time history results of the bridge for the selected earthquakes were analysed and maximum stresses are observed at the deck joint. Separation of deck joints due to these type of earthquakes are possible as per the results since the gap between the two girders at the pier, makes a week connection to the deck. This has created higher stresses at the joint. Maximum displacements are observed at the mid span of the deck joints as shown in Figure 6 and principal stresses in the transverse direction of the bridge is shown in Figure 7. Since earthquakes occur in Australia is not very severe in magnitude compared to other countries in the world, generally these types of repairable damages are anticipated. Therefore strengthening of deck joints and retrofitting of these joints are important in typical girder concrete bridges

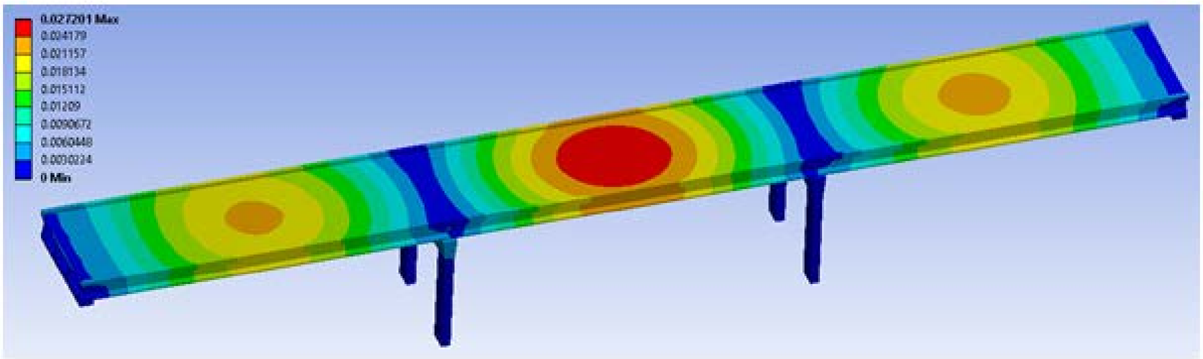


FIGURE 6: DISPLACEMENT CONTOURS OF THE BRIDGE DUE TO EARTHQUAKES

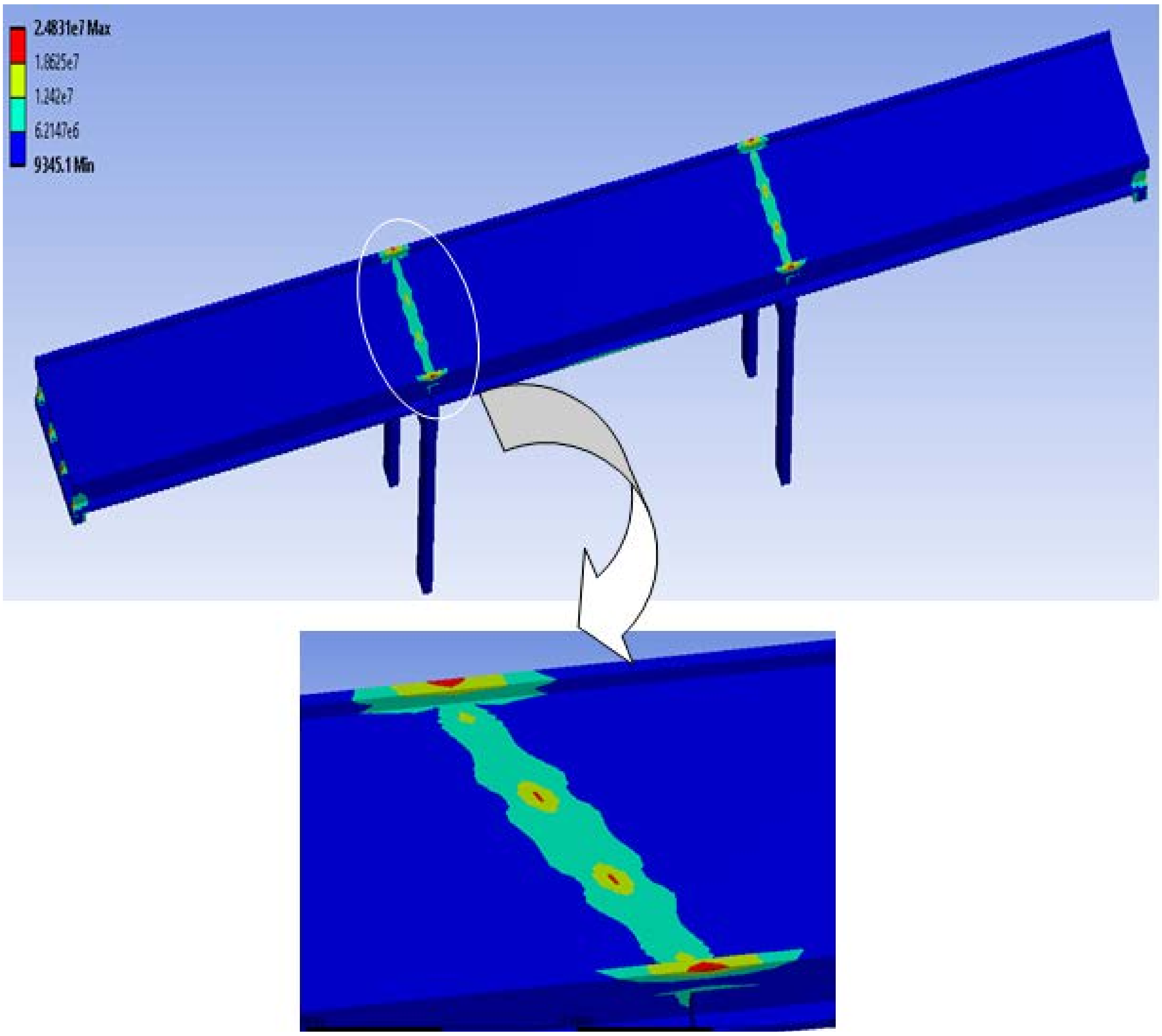


FIGURE 7: MAXIMUM STRESSES AT THE DECK JOINT DUE TO A TYPICAL EARTHQUAKE

Development of fragility curves incorporates the uncertainties in the estimation of ground motion, seismic demand and seismic capacity of the bridges. these curves displays the probability that a bridge is being damaged beyond a specified damage state for various levels of ground shaking. These results can be used for post disaster management planning procedures.

For the selected bridge type, 24 earthquake samples were analysed, and damage stage of the bridge for each of the earthquake acceleration was determined.

The current study considers the damages in the most vulnerable component which was obtained as the deck joint. The damage due to all the earthquake accelerations shows minor damages in the bride at the deck joint and the capacity and demand ratios of the slab deck was used to obtain the fragility curves. Table 2 shows the percentage of damage in each of the peak ground acceleration obtained from the results.

TABLE 2: FRAGILITY DATA FOR THE SELECTED BRIDGE TYPE

Damage state	**PGA**								
	0.07	**0.08**	**0.1**	**0.11**	**0.13**	**0.14**	**0.15**	**0.16**	**0.17**
No Damage	0.208	0.375	0.458	0.708	0.750	0.792	0.875	0.958	1.000

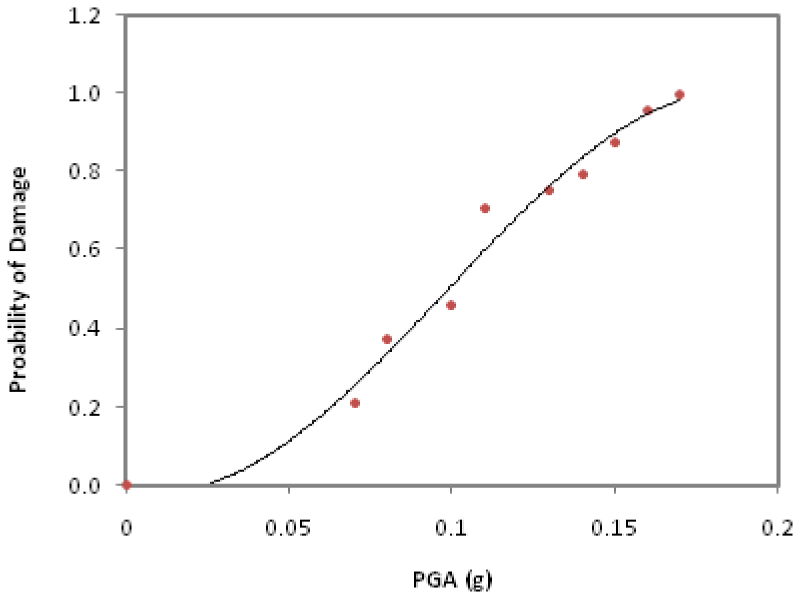


FIGURE 8 : FRAGILITY CURVE

These fragility curves can be used to determine potential losses due to future earthquakes, prioritisation techniques and post disaster decisions. The vulnerability of concrete girder bridges due to typical earthquake time histories are not very severe as per the results. However some retrofitting techniques to be introduced at deck joints to improve the strength of these sections.

After the review on the fragility theory and its historical development, an example of how this study derives fragility function is shown. Through this process, it is clearly illustrated that the high reliability of fragility method comes with a price of theoretical and computational efforts. Similarly, among different fragility deriving methods, assumptions reduce the computational demands but meanwhile induce vulnerability to the method.

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