Evaluation of a case study concrete bridge in Victoria under effect of bushfire

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Abstract

Australia's variable climate has always been a factor in natural disasters that have had significant impacts on its evolving road infrastructure as well as the communities which rely on the roads. Bushfires as one of the major natural disasters have impacted the country directly and indirectly. Bureau of Infrastructure, Transport and Regional Economics (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). Road network plays a vital role not only during the bushfires, but also after the natural hazards' events. As many roads are built across rivers, valleys and other roads, bridges become important part of the transportation network. The paper explores a case study of a Victorian concrete bridge in case of a fire exposure. The Isotherm method has been used for the assessment of case study bridge members where fire exposure duration is a variable in the study. Structural properties are evaluated in bridge components at different locations, during and after the extreme heat. Risk of failure has been evaluated and repair strategies recommended accordingly. The case study presented 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 alternative strategies. Furthermore, emergency services can be informed of the potential risks of using the road network in response time of bushfire events.

Keywords: Bridge, Concrete, Fire Exposure, Risk, Strength Reduction.

1. 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).

Road networks play a vital role not only during the bushfires, but also after the fire events. As many roads are built across rivers, valleys and other roads, they are known as lifeline infrastructure which provides a critical link in the transport network.

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.

This paper explores available methods and recommendations for assessment of concrete bridge structures under the extreme temperature of fire. 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 the bridge 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. A 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).

2. 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 500°C 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.

2.1. 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

2.2. 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:

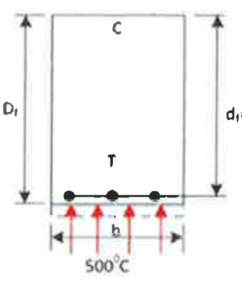
- Concrete spalling
- Concrete cracking
- Concrete delamination
- Compressive strength reduction
- Steel reinforcement and prestressed strands strength reduction

3. Methodology

500°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.

3.1 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 1(a) shows the reduced cross section of reinforced concrete slab fire exposure on one side while Figure 1(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.



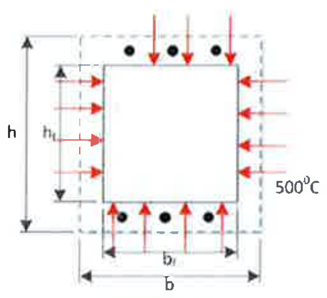
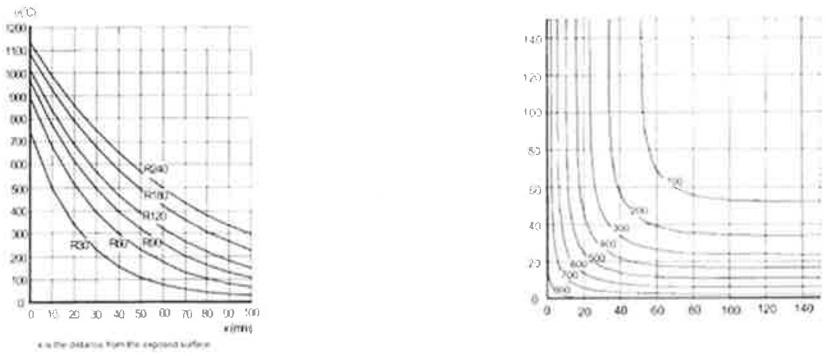


Figure 1: Reduced cross section of reinforced concrete members

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 2: Temperature profiles

3.2 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.

3.3 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.

4. Case study

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 3: Case study bridge

The bridge was constructed in 1958 and consists of reinforced concrete columns, diaphragms and 500mm deep deck slab (Figure 3). 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.

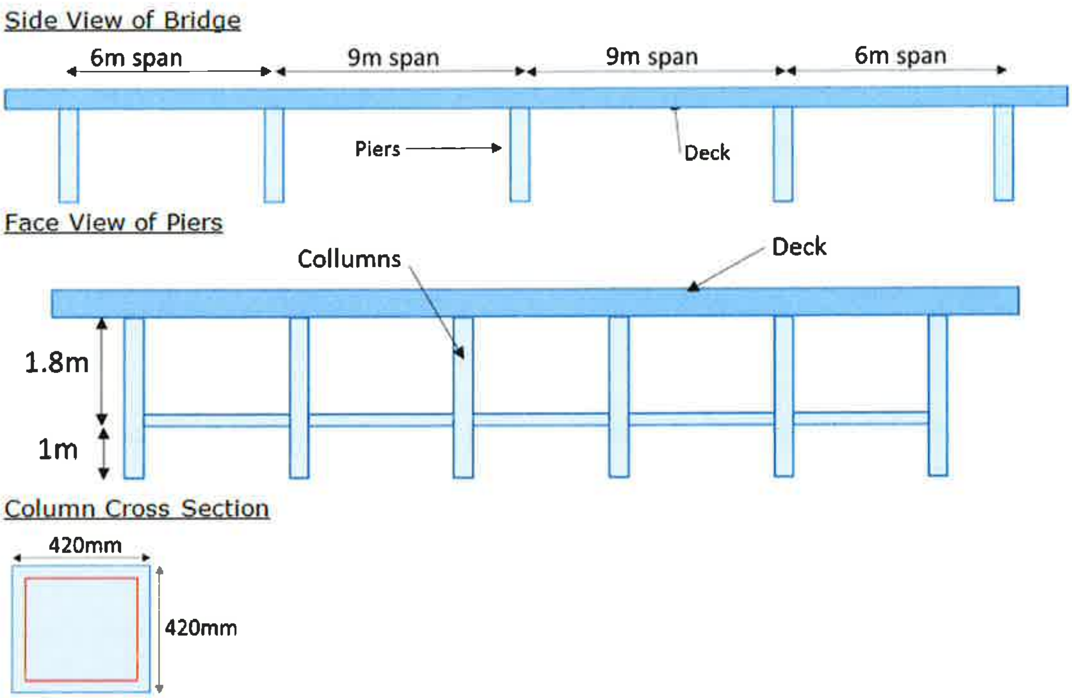


Figure 4: Dimensions of SN5577

5. Analysis

5.1. Deck slabs and deck units

Table 1 shows 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 2 for concrete deck slabs.

Table 1: Depth of 500°C isotherm, and concrete reduction coefficient, K_c 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 2: Temperature of reinforcement and associated yield strength reduction factor(r), and residual yield strength reduction factor ($r_{residual}$).

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

Table 3: 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

5.2. Columns

The depth of the Isotherm 500 and the corresponding concrete strength reduction coefficient in columns are given in Table 4. Tables 5 illustrates bending strength reduction factors, compression capacity reduction factor and the stiffness reduction factor for during and after the fire exposure on columns.

Table 4: Depth of 500°C isotherm at and concrete reduction coefficient, K_c 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 5: 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.

	B(mm)	D(mm)	d(mm)	Mu factor During Fire	Mu factor After Fire	radius of gyration	Le/r	N Factor During Fire	N Factor 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

6. Results

The following table (Table 6) 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 6: Damage and repair requirements

Exposure Time	Deck Units	Columns
30 minutes	500°C isotherm 10mm deep + cracking. Post fire yield strength of reinforcement is unaffected. Repairing of damaged concrete required.	500°C isotherm 16mm deep + 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.

6.1. Risk of failure of Bridges

Based on the structural capacity reductions calculated in Section 5, failure risks of the components have been suggested in Table 7.

Table 7: 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%. No risk of failure.	Moment capacity has dropped by 4%, compression capacity has dropped by 12%, and stiffness has 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.

7. Conclusions

The paper explored 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 duraiton 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.

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