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ANALYSIS OF DESIGN STANDARDS AND APPLIED LOADS ON ROAD STRUCTURES UNDER EXTREME EVENTS

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1. INTRODUCTION

This is the fourth 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 3.2.2 "Analysis of design standards completed" and 3.2.3 "Draft report 4– Loads applied on structures under extreme events (flood, earthquake, fire)", which are due on 30 December 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).

The following draft report presents an analysis of relevant design codes in regards to bridges, culverts and flood-ways design considerations under natural hazards (earthquake, flood and bushfire). Although effort has been made to include major design codes, the main focus of the practice code analysis has been Australian codes, major American codes and European codes. Section 5 also discusses the strengthening methods for reinforced concrete members under natural hazards.

2. EARTHQUAKE LOAD ON STRUCTURES

2.1.DESIGN STANDARDS ANALYSIS

2.1.1. BRIDGES

2.1.1.1. Introduction

Bridges are essential part of the transportation system worldwide, as the closure of important bridges due to damage or collapse in the event of an earthquake can disrupt the total transportation network. There is a strong correlation between the occurrence of major earthquakes and advances in seismic design . Earthquakes have caused significant damages on bridges and lessons are learnt from each earthquake. Heavy damage and catastrophic collapses due to recent earthquakes, have shown that continuing refinements to code philosophy , design procedures and constriction practices need more attention [1]. However constant review of standards and insight into new aspects in seismic analysis and design of bridges have captured these modifications and improved the understanding of the practicing professionals. Europe and Unites States of America are among the major countries that have made significant contribution in this area in the world.

In 1994, European Committee for Standardization (CEN) approved Euro Code 8 -Part 2 ""Earthquake resistant design of Bridges" [2] as the European standard for seismic design of bridges. In United States, there are two national specifications for bridge design published by American Association of State highway and Transpiration officials (AASHTO). They are "Standard Specifications for highway bridges- Division I-A: Seismic design" [3] and :LRFD Bridge design specifications" [4]. In order to evaluate and review the current design standards other than Australian standards, these two overseas standards are selected and described the features in detail in this report.

2.1.1.2. Australian codes (AS 5100 and AS 1170.4)

The design of bridges in Australia is carried out as per the Standard AS 5100 [5] together with AS 1170.4 [6] which is "Earthquake actions in Australia" for seismic actions. Some parameters from AS 1170.4 [6] are used in design of bridges under seismic loads where they are specifically referred. The bridge code AS 5100 [5] is applicable for conventional type of bridges such as slabs, beams, box girders and truss type bridges with span less than 100m. For all other types, specialist advice to be sought for the assessment of seismic effects.

The seismic design rules in Australian Standard for Bridge Design (ASBD) 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.

TABLE 2.1: BRIDGE EARTHQUAKE DESIGN CATEGORY (BEDC) IN AS 5100 [5]

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

2.1.1.3. Euro code (EC 8- Part 2)

The design of bridges for seismic loads in carried out as per the Euro code EC8 of part 2 [2] as described earlier. This Part primarily covers the seismic design of bridges in which the horizontal seismic actions are mainly resisted through bending of the piers or at the abutments; i.e. of bridges collapsed of vertical or nearly vertical pier systems supporting the traffic deck superstructure. It is also applicable to the seismic design of cable-stayed and arched bridges, although its provisions should not be considered as fully covering cases. Suspension bridges, timber and masonry bridges, moveable bridges and floating bridges are not included in the scope of this Part.

There are two basic requirements given in EC 8. They are,

Non – collapse requirement

After the occurrence of the design seismic event, the bridge should retain its structural integrity and adequate residual resistance, although at some parts of the bridge considerable damage may occur. The bridge should be damage-tolerant i.e. those parts of the bridges susceptible to damage, by their contribution to energy dissipation during the design seismic event, should be designed in such a manner as to ensure that, following the seismic event, the structure can sustain the actions from emergency traffic, and inspections and repair can be performed easily.

Minimisation of damage

Only secondary components and those parts of the bridge intended to contribute to energy dissipation during the design life of the bridge should incur minor damage during earthquakes with a high probability of occurrence. The non-collapse requirement for bridges under the design seismic event is more stringent than the relevant requirement for buildings, as it contains the continuation of emergency traffic.

The bridge shall be designed so that its behaviour under the design seismic action is either ductile, or limited ductile/essentially elastic, depending on the seismicity of the site, on whether seismic isolation is adopted for its design, or any other constraints which may prevail. This behaviour (ductile or limited ductile) is characterised by the global force-displacement relationship of the structure, shown in FIGURE 2.1.

• ductile behaviour, corresponding to values of the behaviour factor 1.50<q≤ 3.5

• limited ductile behaviour, corresponding to q-values ≤ 1.50

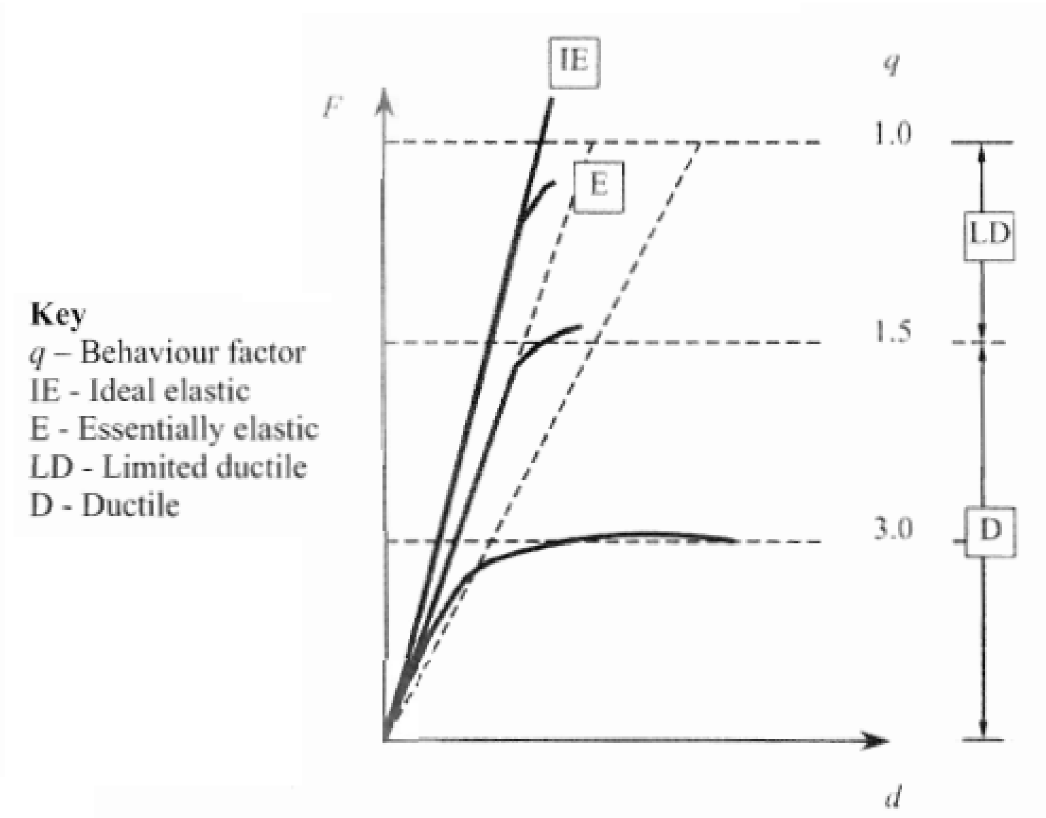


FIGURE 2.1: SEISMIC BEHAVIOUR

2.1.1.4. US design guidelines

As per AASHTO guidelines [4], bridges shall be designed to have a low probability of collapse but may suffer significant damage and disruption to service when subject to earthquake ground motions that have a seven percent probability of exceedance in 75 yr. Partial or complete replacement may be required.

Seismic Hazard

The seismic hazard at a bridge site shall be characterized by the acceleration response spectrum for the site and the site factors for the relevant site class. The acceleration spectrum shall be determined using either the General Procedure or the Site Specific Procedure described as below. A Site-Specific Procedure shall be used if any one of the following conditions exist:

• The site is located within 6 mi. of an active fault,

• The site is classified as Site Class F

• Long-duration earthquakes are expected in the region,

• The importance of the bridge is such that a lower probability of exceedance (and therefore a longer return period) should be considered.

If time histories of ground acceleration are used to characterize the seismic hazard for the site, they shall be determined in accordance with the guideline.

General Procedure

The General Procedure shall use the peak ground acceleration coefficient (PGA) and the short- and long period spectral acceleration coefficients (SS and S1 respectively) to calculate the spectrum as specified in FIGURE 2.4. Values of PGA, SS and S1 shall be determined from either Figures 3.10.2.1-1 to 3.10.2.1-21 of AASHTO LRFD [4] as appropriate, or from state ground motion maps approved by the Owner. Linear interpolation shall be used for sites located between contour lines or between a contour line and a local maximum or minimum.

Values for the coefficients PGA, SS and S1 are expressed in percent in Figures 3.10.2.1-1 to 3.10.2.1-21 in AASHTO LRFD [4]. Numerical values are obtained by dividing contour values by 100. Local maxima and minima are given inside the highest and lowest contour for a particular region. The above coefficients are based on a uniform risk model of seismic hazard. The probability that a coefficient will not be exceeded at a given location during a 75-yr period is estimated to be about 93 percent, i.e., a seven percent probability of exceedance. The use of a 75-yr interval to characterize this probability is an arbitrary convenience and does not imply that all bridges are thought to have a useful life of 75 yr. It can be shown that an event with the above probability of exceedance has a return period of about 1,000 yr and is called the design earthquake. Larger earthquakes than that implied by the above set of coefficients have a finite probability of occurrence throughout the United States.

Site Specific Procedure

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required as described above and may be performed for any site. The objective of the site-specific probabilistic ground-motion analysis should be to generate a uniform-hazard acceleration response spectrum considering a seven percent probability of exceedance in 75 yr for spectral values over the entire period range of interest. This analysis should involve establishing:

• The contributing seismic sources;

• An upper-bound earthquake magnitude for each source zone;

• Median attenuation relations for acceleration response spectral values and their associated standard deviations;

• A magnitude-recurrence relation for each source zone; and

• A fault-rupture-length relation for each contributing fault.

2.1.2. FLOOD-WAYS AND CULVERTS

Floodways and culverts are very important features in road infrastructure. It is widely accepted in the community that these type of structures have good performance under earthquake loading and that such structures are able to accommodate the deflections imposed by the ground vibrations without failure. There are cases of structural failure and total collapse of such structures due to seismic events.

Clough and Fragaszy ,1977 [7] observed concrete cantilever walls supporting open channel floodways that had collapsed where peak ground accelerations were 0.5g or more in the 1971 San Fernando earthquake. However, in that case, soil conditions were good. All of these wall cases where collapse or severe damage/deformations occurred are well outside of the conditions and situations.

However there isn't any available design guidelines for analysis and design of floodways and culverts for seismic actions rather than earth pressure loads on these type of structures in the literature. Therefore in this report, only design procedures, design loads and combinations and some special requirements such as detailing for different types of bridge structures are described in details as per the design standards.

2.2. DESIGN LOADS & LOAD COMBINATIONS

2.2.1. BRIDGES

2.2.1.1. Australian Code (AS 5100)

Horizontal earthquake force is determined in the principal axis or the major orthogonal direction of the bridge. The total horizontal earthquake load can be applied at a vertical level corresponds to the mass centroid of the bridge deck. The design load shall be distributed along the length of the bridge according to the mass distribution along the bridge deck.

The vertical earthquake load shall be considered and applied independent of the horizontal load.

Horizontal earthquake force

The horizontal design earthquake force H_u^* shall be determined from the following equation

$$H_u^* = I\left(\frac{\mathrm{CS}}{\mathrm{R_f}}\right)\mathrm{G_g} \tag{2.1}$$

within the limits

$$H_u^* \geq 0.02\mathrm{G_g} \tag{2.2}$$

and

$$H_u^* = I\left(\frac{2.5\mathrm{a}}{\mathrm{R_f}}\right)\mathrm{G_g} \tag{2.3}$$

Where

I = importance factor

C = earthquake design coefficient

S = site factor

Rf = structural response factor

Gg = total unfactored dead load including superimposed dead load

a = acceleration coefficient

Iimportance factor

The importance factor (I) is given in following Table depending on the structure type.

TABLE 2.2: IMPORTANCE FACTOR

Structure type	**Importance factor, I**
III	1.25
II and I	1.00

Earthquake design coefficient

The earthquake design coefficient (C) is determined for each horizontal and vertical direction separately from the following equation.

$$C = \frac{1.25a}{T^{2/3}} \tag{2.4}$$

where T (in seconds) is the structure period of the first dominant mode of free vibration in the direction under consideration.

The structure period (T) to be determined by structural analysis based on a recognized theoretical approach.

For bridge structures in BEDC-1 only, T may be approximated from,

$$T = 0.063\sqrt{\delta} \tag{2.5}$$

where δ is the displacement under self-weight, in millimetres, with gravity applied in the direction of interest, i.e., horizontal or vertical

For bridge structures in BEDC-1 only and with a more general mass distribution, T may be approximated from,

$$T = 2\pi\sqrt{\frac{\sum m_i(\delta_i)^2}{g\sum(m_i\delta_i)}} \tag{2.6}$$

The structure is represented by a number of discrete masses (m_i) in kilograms, while δi, in metres, is the deflection at the centroid of mass (m_i) due to a force of ($m_i g$) applied at the centroid in the direction of interest.

Site factor

The site factor (S) shall be as specified in AS 1170.4 [6] for the appropriate soil profile below the founding level. The soil profile shall be established from geotechnical data and classified in accordance with AS 1726 [8]. Interpolation for soil profiles in between those given in AS 1170.4 [6] is permitted.

The spectral shape factor given in AS1170.4 [6] for the appropriate site sub-soil class is given as follows.

TABLE 2.3: SITE FACTOR FOR DIFFERENT SOIL TYPES

	Site sub-soil class				
Period (seconds)	A_e Strong rock	B_e Rock	C_e Shallow soil	D_e Deep or soft soil	E_e Very soft soil
0.0	2.35 (0.8)*	2.94 (1.0)*	3.68 (1.3)*	3.68 (1.1)*	3.68 (1.1)*
0.1	2.35	2.94	3.68	3.68	3.68
0.2	2.35	2.94	3.68	3.68	3.68
0.3	2.35	2.94	3.68	3.68	3.68
0.4	1.76	2.20	3.12	3.68	3.68
0.5	1.41	1.76	2.50	3.68	3.68
0.6	1.17	1.47	2.08	3.30	3.68
0.7	1.01	1.26	1.79	2.83	3.68
0.8	0.88	1.10	1.56	2.48	3.68
0.9	0.78	0.98	1.39	2.20	3.42
1.0	0.70	0.88	1.25	1.98	3.08
1.2	0.59	0.73	1.04	1.65	2.57
1.5	0.47	0.59	0.83	1.32	2.05
1.7	0.37	0.46	0.65	1.03	1.60
2.0	0.26	0.33	0.47	0.74	1.16
2.5	0.17	0.21	0.30	0.48	0.74
3.0	0.12	0.15	0.21	0.33	0.51
3.5	0.086	0.11	0.15	0.24	0.38
4.0	0.066	0.083	0.12	0.19	0.29
4.5	0.052	0.065	0.093	0.15	0.23
5.0	0.042	0.053	0.075	0.12	0.18
Equations for spectra					
$0 < T \le 0.1$	$0.8 + 15.5T$	$1.0 + 19.4T$	$1.3 + 23.8T$	$1.1 + 25.8T$	$1.1 + 25.8T$
$0.1 < T \le 1.5$	$0.704/T$ but ≤ 2.35	$0.88/T$ but ≤ 2.94	$1.25/T$ but ≤ 3.68	$1.98/T$ but ≤ 3.68	$3.08/T$ but ≤ 3.68
$T > 1.5$	$1.056/T^2$	$1.32/T^2$	$1.874/T^2$	$2.97/T^2$	$4.62/T^2$

* Values in brackets correspond to values of spectral shape factor for the modal response spectrum and the numerical integration time history methods and for use in the method of calculation of forces on parts and components

Structural response factor

The structural response factor (R_f) is the minimum value given in TABLE 2.4 for the appropriate bridge structural system.

TABLE 2.4: STRUCTURAL RESPONSE FACTOR

Bridge structural system	Structural response factor (Rf)
Piers and deck form a continuous frame to resist horizontal earthquake force	6.0
Deck continuous over piers, supported on bearings	5.0
Bridges with single column piers to resist horizontal earthquake force	3.5
Bridges with simply supported spans	3.0

Acceleration coefficient

The acceleration coefficient (a) is specified in AS 1170.4 [6] and earthquake hazard factor is equivalent to an acceleration coefficient with an annual probability of exceedance in 1/500, (i.e., a 10% probability of exceedance in 50 years)(AS 1170.4). The hazard factor (Z) shall be taken from TABLE 2.5 or, where the location is not listed, be determined from Figures 3.2(A) to 3.2(F) of AS 1170.4 [6]. A general overview of the hazard factor (Z) for Australia is shown in FIGURE 2.2.

TABLE 2.5: HAZARD FACTOR (Z) FOR SPECIFIC AUSTRALIAN LOCATIONS

Location	Z	Location	Z	Location	Z
Adelaide	0.10	Geraldton	0.09	Port Augusta	0.11
Albany	0.08	Gladstone	0.09	Port Lincoln	0.10
Albury/Wodonga	0.09	Gold Coast	0.05	Port Hedland	0.12
Alice Springs	0.08	Gosford	0.09	Port Macquarie	0.06
Ballarat	0.08	Grafton	0.05	Port Pirie	0.10
Bathurst	0.08	Gippsland	0.10	Robe	0.10
Bendigo	0.09	Goulburn	0.09	Rockhampton	0.08
Brisbane	0.05	Hobart	0.03	Shepparton	0.09
Broome	0.12	Karratha	0.12	Sydney	0.08
Bundaberg	0.11	Katoomba	0.09	Tamworth	0.07
Burnie	0.07	Latrobe Valley	0.10	Taree	0.08
Cairns	0.06	Launceston	0.04	Tennant Creek	0.13
Camden	0.09	Lismore	0.05	Toowoomba	0.06
Canberra	0.08	Lorne	0.10	Townsville	0.07
Carnarvon	0.09	Mackay	0.07	Tweed Heads	0.05
Coffs Harbour	0.05	Maitland	0.10	Uluru	0.08
Cooma	0.08	Melbourne	0.08	Wagga Wagga	0.09
Dampier	0.12	Mittagong	0.09	Wangaratta	0.09
Darwin	0.09	Morisset	0.10	Whyalla	0.09
Derby	0.09	Newcastle	0.11	Wollongong	0.09
Dubbo	0.08	Noosa	0.08	Woomera	0.08
Esperance	0.09	Orange	0.08	Wyndham	0.09
Geelong	0.10	Perth	0.09	Wyong	0.10
Meckering region				**Islands**	
Ballidu	0.15	Meckering	0.20	Christmas Island	0.15
Corrigin	0.14	Northam	0.14	Cocos Islands	0.08
Cunderdin	0.22	Wongan Hills	0.15	Heard Island	0.10
Dowerin	0.20	Wickepin	0.15	Lord Howe Island	0.06
Goomalling	0.16	York	0.14	Macquarie Island	0.60
Kellerberrin	0.14			Norfolk Island	0.08

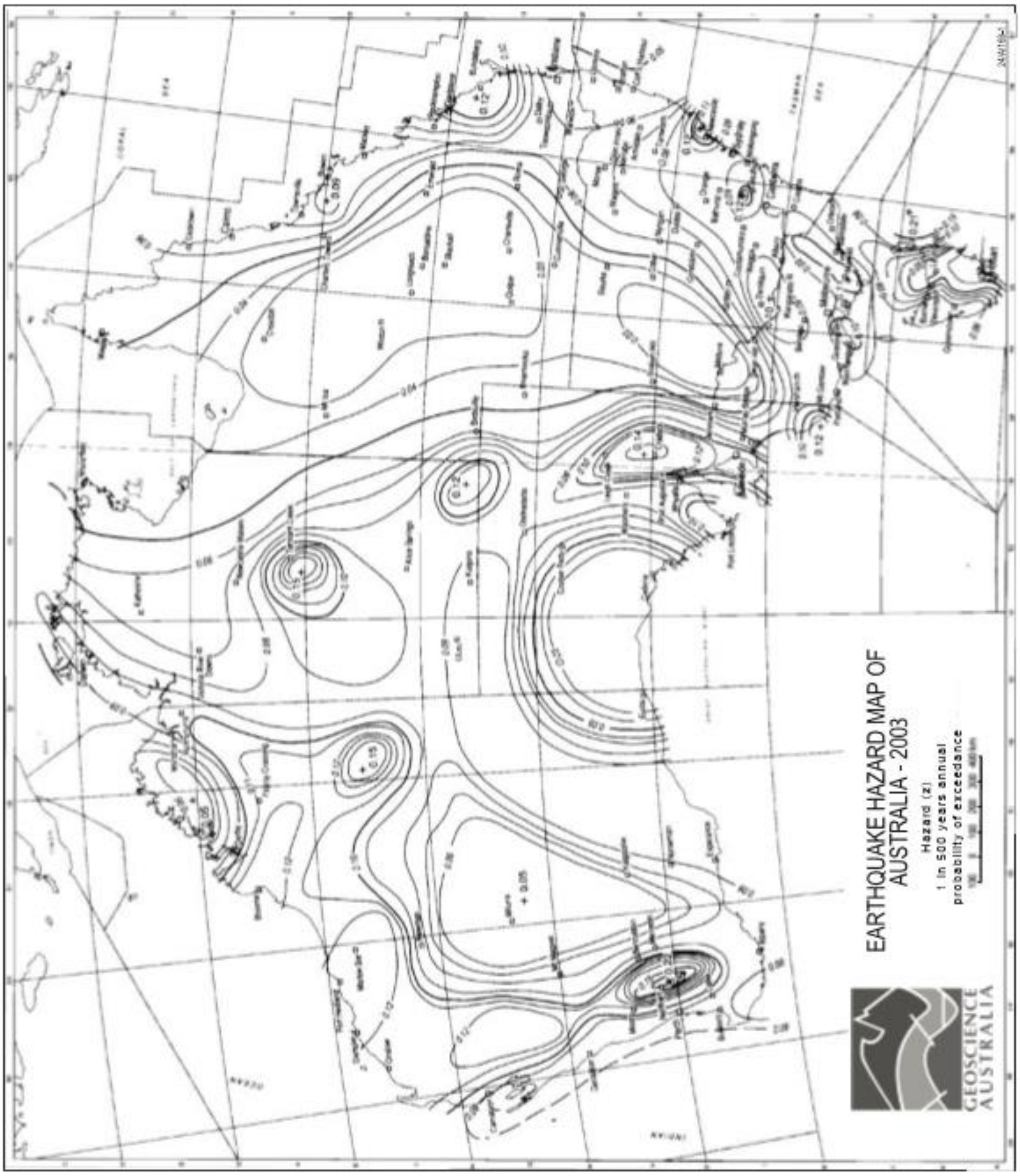


FIGURE 2.2: HAZARD FACTOR (Z)

Vertical earthquake force

The total vertical design earthquake force, acting either up or down, to be determined using the same procedures described above considering the structure period of the dominant mode of free vibration in the vertical direction. The vertical design force should not be less than 50% of the maximum horizontal design earthquake force in either direction. The vertical design earthquake force does not include normal gravity force. Vertical earthquake forces need be applied to the structure in accordance with the distribution of mass. The distribution of forces between the superstructure and the substructure shall be in accordance with the stiffness of the bearings or connections. Where vertical earthquake forces do not produce adverse critical effects, they can be ignored.

2.2.1.2. Euro code (EC 8- Part 2)

In general only the three translational components of the seismic action need to be taken into account for the design of bridges as per EC 8 [9]. Each component of earthquake motion shall be quantified in terms of a response spectrum, or a time-history representation which are discussed in Section 2.3. This motion at a given point on the surface represented by an elastic ground acceleration response spectrum is called an "elastic response spectrum. When the response spectrum method is applied, the bridge may analysed separately for the translational components of the seismic action in the longitudinal, transverse and vertical directions. In this case the seismic action is represented by three one-component actions, one for each direction. When non-linear time-history analysis is performed, the bridge shall be analysed under the simultaneous action of the different components. The seismic action is applied at the interface between the structure and the ground. Springs can be used to represent the soil stiffness either in connection with spread footings or with deep foundations, such as piles, shafts (caissons), etc.

Horizontal elastic response spectrum (EC8-1)

For the horizontal components of the seismic action, the elastic response spectrum Se(T) is defined by the following expressions.

$$0 \le T \le T_B : \mathrm{S_e(T)} = \alpha_g S\left[1 + \frac{\mathrm{T}}{\mathrm{T_B}}(\eta * 2.5 - 1)\right] \tag{2.7}$$

$$T_B \le T \le T_C : \mathrm{S_e(T)} = \alpha_g S[(\eta * 2.5)] \tag{2.8}$$

$$T_C \le \mathrm{T} \le T_D : \mathrm{S_e(T)} = \alpha_g * \eta * \mathrm{S} * 2.5\left[\frac{\mathrm{T_C}}{\mathrm{T}}\right] \tag{2.9}$$

$$T_D \le \mathrm{T} \le 4\mathrm{s} : \mathrm{S_e(T)} = \alpha_g * \eta * \mathrm{S} * 2.5\left[\frac{\mathrm{T_C T_D}}{\mathrm{T^2}}\right] \tag{2.10}$$

S_e (T) ordinate of the elastic response spectrum, T vibration period of a linear single-degree-of-freedom system, a_g design ground acceleration T_B, T_C limits of the constant spectral acceleration branch, T_D value defining the beginning of the constant displacement response range of the spectrum, S soil factor, η damping correction factor.

TABLE 2.6: RECOMMENDED VALUES OF THE PARAMETERS FOR TYPES 1 & 2 ELASTIC RESPONSE SPECTRA

	S		$T_e(S)$		$T_C(S)$		$T_D(S)$		a_{vg}/a_g	
Spectrum type	1	2	1	2	1	2	1	2	1	2
Soil class A	1.0	1.00	0.15	0.05	0.4	0.25	2.0	1.2		
Soil class B	1.2	1.35	0.15	0.05	0.5	0.25	2.0	1.2		
Soil class C	1.15	1.50	0.20	0.10	0.6	0.25	2.0	1.2		
Soil class D	1.35	1.80	0.20	0.10	0.8	0.30	2.0	1.2		
Soil class E	1.4	1.60	0.15	0.05	0.5	0.25	2.0	1.2		

Two types of response spectra are defined. Type 2 spectrum is recommended only for regions where the design earthquake has a surface has a surface wave magnitude Ms ≤ 5.5. The elastic displacement response spectrum, $DS_e(T)$, is obtained by direct transformation of the elastic acceleration spectrum, $S_e(T)$, using the following expression:

$$DS_e(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \qquad (2.11)$$

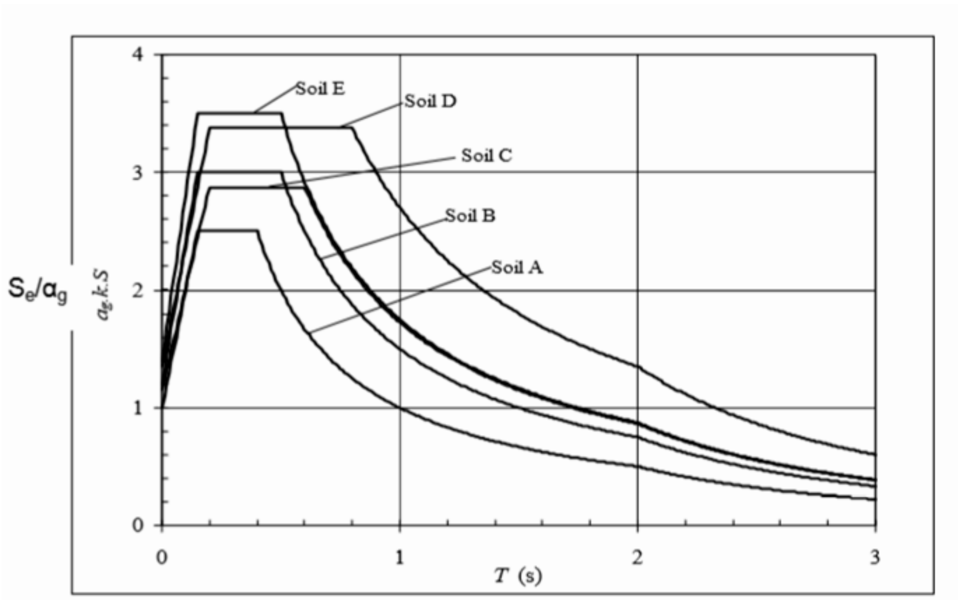


FIGURE 2.3: RECOMMENDED ELASTIC RESPONSE SPECTRUM, TYPE 1

When the vertical component of the seismic motion needs to be taken into account, the site-dependent response spectrum of this component shall be taken in accordance with EN 1998-1 :2004 [9]. The vertical component of the seismic action shall be represented by an elastic response spectrum, $S_{ve}(T)$, derived using following expressions.

$$0 \le T \le T_B : \mathrm{S_{ve}(T)} = \mathrm{a_{vg}}\left[1 + \frac{\mathrm{T}}{\mathrm{T_B}}(\eta * 3.0 - 1)\right] \qquad (2.12)$$

$$T_B \leq T \leq T_C : S_{ve}(T) = a_{vg}[(\eta * 3.0)] \tag{2.13}$$

$$T_C \leq T \leq T_D : S_{ve}(T) = a_{vg} * \eta * 3.0 \left[\frac{T_C}{T}\right] \tag{2.14}$$

$$T_D \leq T \leq 4s : S_{ve}(T) = a_{vg} * \eta * 3.0 \left[\frac{T_C T_D}{T^2}\right] \tag{2.15}$$

NOTE : The values to be ascribed to T_B, T_C, T_D and avg for each type (shape) of vertical spectrum to be used in a country may be found in its National Annex. The recommended choice is the use of two types of vertical spectra: Type 1 and Type 2. As for the spectra defining the horizontal components of the seismic action, if the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, Ms, not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters describing the vertical spectra are given in Table 3.4. These recommended values do not apply for special ground types S1 and S2.

TABLE 2.7: RECOMMENDED VALUES OF PARAMETERS DESCRIBING THE VERTICAL ELASTIC RESPONSE SPECTRA

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0.9	0.05	0.15	1.0
Type 2	0.45	0.05	0.15	1.0

Design spectrum for elastic analysis (EC8-1)

The horizontal components are defined by Eqs. (2.7) to (2.10) by replacing the damping correction factor η by the inverse of the behaviour factor q (i.e. using η=1/q). For the very short period range following equation replaces.

$$0 \leq T \leq T_B: S_d(T) = \alpha_g S \left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \tag{2.16}$$

US Design guidelines

The behavior of a bridge during an earthquake is strongly related to the soil conditions at the site. Soils can amplify ground motions in the underlying rock, sometimes by factors of two or more. The extent of this amplification is dependent on the profile of soil types at the site and the intensity of shaking in the rock below. Sites are classified by type and profile for the purpose of defining the overall seismic hazard, which is quantified as the product of the soil amplification and the intensity of shaking in the underlying rock.

Site Class Definitions

A site shall be classified as A though F in accordance with the site class definitions in TABLE 2.8. Sites shall be classified by their stiffness as determined by the shear wave velocity in the upper 100 ft. Standard Penetration Test (SPT), blow counts and undrained shear strengths of soil samples from soil borings may also be used to classify sites as indicated in Table TABLE 2.8.

TABLE 2.8: SITE CLASS DEFINITION

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, V_s> 5000ft/s
B	Rock with 2,500 ft/sec <V_s<5000ft/s
C	Very dense soil and soil rock with 1,200 ft/s < V_s < 2,500 ft/s or with either N > 50 blows/ft, or S_U > 2.0 ksf
D	Stiff soil with 600 ft/s < V_s < 1,200 ft/s, or with either 15 < N < 50 blows/ft, or 1.0 < S_U < 2.0 ksf
E	Soil profile with V_s < 600 ft/s or with either N < 15 blows/ft or S_U < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40 percent and S_U<0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays (H > 10 ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H >120 ft)

Site Factors

Site Factors F_{pga}, F_a and F_v specified in Tables TABLE 2.9, TABLE 2.10 and TABLE 2.11 can be used in the zero-period, short-period range, and long-period range, respectively. These factors shall be determined using the Site Class given in Table 2.8 and the mapped values of the coefficients PGA, SS , and S1 in Figures 3.10.2.1-1 to 3.10.2.1-21 in AASHTO [4].

TABLE 2.9: VALUES OF SITE FACTOR, F_{PGA} AT ZERO- PERIOD ON ACCELERATION SPECTRUM

Site class	Peak ground acceleration coefficient				
	PGA<0.10	PGA=0.20	PGA=0.30	PGA =0.40	PGA>0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

TABLE 2.10: VALUES OF SITE FACTOR, F_A FOR SHORT- PERIOD RANGE OF ACCELERATION SPECTRUM

Site class	Spectral acceleration coefficient at period 0.2 sec(S_S)[1]				
	S_S<0.25	S_S =0.50	S_S =0.75	S_S =1.0	S_S >1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

TABLE 2.11: VALUES OF SITE FACTOR, F_V, FOR LONG PERIOD RANGE OF ACCELERATION SPECTRUM

Site class	Spectral acceleration coefficient at period 1.0sec(S_1)[1]				
	S_1<0.1	S_1 =0.2	S_1 =0.3	S_1 =0.4	S_1 >0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]	*	*	*	*	*

Site Class B (soft rock) is taken to be the reference site category for the USGS and NEHRP MCE ground shaking maps. Site class B rock is therefore the site condition for which the site factor is 1.0. Site classes A, C, D, and E have separate sets of site factors for zero-period (F_{pga}), the short period range (F_a) and long-period range (F_v), as indicated in Tables TABLE 2.9, TABLE 2.10 and TABLE 2.11. These site factors generally increase as the soil profile becomes softer (in going from site class A to E). Except for site class A (hard rock), the factors also decrease as the ground motion level increases, due to the strongly nonlinear behavior of the soil. For a given site class, C, D, or E, these nonlinear site factors increase the ground motion more in areas having lower rock ground motions than in areas having higher rock ground motions

Design Response Spectrum

The five-percent-damped-design response spectrum shall be taken as specified in FIGURE 2.4. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients from Figures 3.10.2.1-1 to 3.10.2.1-21 of AASHTO [4], scaled by the zero-, short-, and long-period site factors, F_{pga} , F_a , and F_v , respectively.

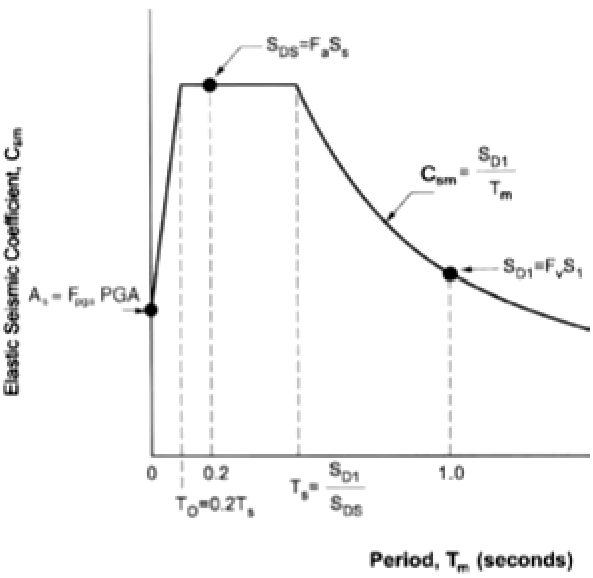


FIGURE 2.4: DESIGN RESPONSE SPECTRUM

The long-period portion of the response spectrum in FIGURE 2.4 is inversely proportional to the period, T. In the previous edition of these Specifications, this portion of the spectrum was inversely proportional to T2/3. The consequence of this change is that spectral accelerations at periods greater than 1.0 s are smaller than previously specified (for the same ground acceleration and soil type), and greater than previously specified for periods less than 1.0 s (but greater than TS). This change is consistent with the observed characteristics of response spectra calculated from recorded ground motions. For periods exceeding about 3 s, it has been observed that in certain seismic environments spectral displacements tend to a constant value which implies that the acceleration spectrum becomes inversely proportional to T_2 at these periods. As a consequence, the spectrum in FIGURE 2.4 and Eq (2.21) may give conservative results for long period bridges (greater than about 3 s).

Elastic Seismic Response Coefficient

For periods less than or equal to T_0, the elastic seismic coefficient for the m^{th} move of vibration, C_{sm}, shall be taken as:

$$C_{sm} = A_S + (S_{DS} – A_S)\ (T_m / T_0) \qquad (2.17)$$

in which

$$A_S = F_{pga}\ PGA \qquad (2.18)$$

$$S_{DS} = F_a\ S_S \qquad (2.19)$$

where:

PGA = peak ground acceleration coefficient on rock (Site Class B)

S_S = horizontal response spectral acceleration coefficient at 0.2-sec period on rock (Site Class B)

T_m = period of vibration of m^{th} mode (s)

T_0 = reference period used to define spectral shape = 0.2 T_S (s)

T_S = corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS} (s)

For periods greater than or equal to T_0 and less than or equal to T_S , the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{DS} \qquad (2.20)$$

For periods greater than T_S , the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{D1} / T_m \qquad (2.21)$$

in which:

$S_{D1} = F_v S_1$ (2.22)

where:

S_1 = horizontal response spectral acceleration coefficient at 1.0 sec period on rock (Site Class B)

Seismic Performance Zones Each bridge shall be assigned to one of the four seismic zones in accordance with TABLE 2.12 TABLE 2.12: SEISMIC ZONES using the value of S_{D1} given by Eq (2.22).

TABLE 2.12: SEISMIC ZONES

Acceleration Coefficient, S_{D1}	**Seismic Zone**
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

Importance classes

Differentiation of target reliability may be effected by means of importance factors γ_I as,

$A_{Ed} = \gamma_1 A_{Ek}$ (2.23)

A_{Ed} is the design seismic action and A_{Ek} is the characteristic seismic action (usually corresponding to a return period of 475 years). The recommended importance classes and corresponding factors as shown in TABLE 2.13.

TABLE 2.13: BRIDGE IMPORTANCE CLASSES

Importance Class		γI
Greater than average	iii	1.3
Average	ii	1.00
Less than average	i	0.85

Response Modification Factors

To apply the response modification factors specified here, the structural details shall satisfy the provisions of AASHTO [4] 5.10.2.2, 5.10.11, and 5.13.4.6. As an alternative to the use of the R-factors, specified in TABLE 2.15 for connections, monolithic joints between structural members and/or structures, such as a column-to-footing connection, may be designed to transmit the maximum force effects that can be developed by the inelastic hinging of the column or multicolumn bent they connect. If an inelastic time history method of analysis is used, the response modification factor, R, shall be taken as 1.0 for all substructure and connections.

TABLE 2.14: RESPONSE MODIFICATION FACTORS—SUBSTRUCTURES

Substructure	Operational Category		
	Critical	Essential	Other
Wall-type piers—larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single column	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical pile only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

TABLE 2.15: RESPONSE MODIFICATION FACTORS—CONNECTIONS

Connection	All Operational Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

Loads Application

Seismic loads shall be assumed to act in any lateral direction. The appropriate R-factor shall be used for both orthogonal axes of the substructure. A wall-type concrete pier may be analyzed as a single column in the weak direction if all the provisions for columns, as specified in Section 5 of AASHTO [3], are satisfied.

Combination of Seismic Force Effects

The elastic seismic force effects on each of the principal axes of a component resulting from analyses in the two perpendicular directions shall be combined to form two load cases as follows:

• 100 percent of the absolute value of the force effects in one of the perpendicular directions combined with 30 percent of the absolute value of the force effects in the second perpendicular direction, and

• 100 percent of the absolute value of the force effects in the second perpendicular direction combined with 30 percent of the absolute value of the force effects in the first perpendicular direction.

Calculation of Design Forces

For single-span bridges, regardless of seismic zone, the minimum design connection force effect in the restrained direction between the superstructure and the substructure need not be less than the product of the acceleration coefficient, A_S , specified in Eq (2.18) , and the tributary permanent load. Minimum support lengths at expansion bearings of multi span bridges shall either comply with AASHTO [4] 4.7.4.4 and dampers shall be provided.

Seismic Zone 1

For bridges in Zone 1 where the acceleration coefficient, A_S, as specified in Eq. Eq (2.18), is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For all other sites in Zone 1, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake. The horizontal design connection force shall be addressed from the point of application through the substructure and into the foundation elements. For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment. If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing. Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in Seismic Zone 1 and all single span bridges, these seismic shear forces shall not be less than the connection force specified herein.

Seismic Zone 2

Structures in Seismic Zone 2 shall be analyzed according to the minimum requirements specified in AASHTO [3] Section 4.7.4.1 and 4.7.4.3. Except for foundations, seismic design forces for all components, including pile bents and retaining walls, shall be determined by dividing the elastic seismic forces, obtained from Section 2.2.1.18, by the appropriate response modification factor, R, specified in Table 2.14.

Seismic design forces for foundations, other than pile bents and retaining walls, shall be determined by dividing elastic seismic forces, obtained from Article 2.2.1.18, by half of the response modification factor, R, from Table 2.14, for the substructure component to which it is attached. The value of R/2 shall not be taken as less than 1.0. Where a group load other than Extreme Event I, specified in Table below, governs the design of columns, the possibility that seismic forces transferred to the foundations may be larger than those calculated using the procedure specified above, due to possible over strength of the columns, shall be considered.

Seismic Zones 3 and 4

Structures in Seismic Zones 3 and 4 shall be analyzed according to the minimum requirements specified in Section 2.3.1.19. The design forces of each component shall be taken as the lesser of those determined using:

Modified Design Forces

Modified design forces shall be determined as specified in Section 2.2.1.21, except that for foundations the R-factor shall be taken as 1.0.

Inelastic Hinging Forces

Where inelastic hinging is invoked as a basis for seismic design, the force effects resulting from plastic hinging at the top and/or bottom of the column shall be calculated after the preliminary design of the columns has been completed utilizing the modified design forces as the seismic loads. The consequential forces resulting from plastic hinging should then be used for determining design forces for most components as identified.

Single Columns and Piers

Force effects shall be determined for the two principal axes of a column and in the weak direction of a pier or bent as follows:

Step 1—Determine the column over strength moment resistance. Use a resistance factor, φ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials, the applied axial load in the column shall be determined using Extreme Event Load Combination I, with the maximum elastic column axial load from the seismic forces determined in accordance with Section 2.2.1.17 taken as EQ.

Step 2—Using the column overstrength moment resistance, calculate the corresponding column shear force. For flared columns, this calculation shall be performed using the overstrength resistances at both the top and bottom of the flare in conjunction with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur, the column length between plastic hinges shall be used to calculate the column shear force. Force effects corresponding to a single column hinging shall be taken as:

• Axial Forces—Those determined using Extreme Event Load Combination I, with the unreduced maximum and minimum seismic axial load of Section 2.2.1.17 taken as EQ.

- Moments—Those calculated in Step 1.
- Shear Force—That calculated in Step 2.

Piers with Two or More Columns

Force effects for bents with two or more columns shall be determined both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent, the forces shall be determined as for single columns as described above. In the plane of the bent, the forces shall be calculated as follows:

Step 1—Determine the column over strength moment resistances. Use a resistance factor, φ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials the initial axial load should be determined using the Extreme Event Load Combination I with EQ = 0.

Step 2—Using the column over strength moment resistance, calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the pier. If a partial-height wall exists between the columns, the effective column height should be taken from the top of the wall. For pile bents, the length of pile above the mud line shall be used to calculate the shear force.

Step 3—Apply the bent shear force to the center of mass of the superstructure above the pier and determine the axial forces in the columns due to overturning when the column over strength moment resistances are developed.

Step 4—Using these column axial forces as EQ in the Extreme Event Load Combination I, determine revised column over strength moment resistance. With the revised over strength moment resistances, calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within ten percent of the value previously determined, use this maximum bent shear force and return to Step 3. The forces in the individual columns in the plane of a bent corresponding to column hinging shall be taken as:

• Axial Forces—The maximum and minimum axial loads determined using Extreme Event Load Combination I, with the axial load determined from the final iteration of Step 3 taken as EQ and treated as plus and minus.

• Moments—The column over strength moment resistances corresponding to the maximum compressive axial load specified above.

• Shear Force—The shear force corresponding to the column over strength moment resistances specified above, noting the provisions in Step 2 above.

Column and Pile Bent Design Forces

Design forces for columns and pile bents shall be taken as a consistent set of the lesser of the forces determined as specified in Section 2.2.1.22, applied as follows:

• Axial Forces—The maximum and minimum design forces determined using Extreme Event Load Combination I with either the elastic design values determined in Section 2.2.1.17 taken as EQ, or the values corresponding to plastic hinging of the column taken as EQ.

• Moments—The modified design moments determined for Extreme Event Limit State Load Combination I.

• Shear Force—The lesser of either the elastic design value determined for Extreme Event Limit State Load Combination I with the seismic loads combined as specified in Section 2.2.1.17 and using an R-factor of 1 for the column, or the value corresponding to plastic hinging of the column.

Pier Design Forces

The design forces shall be those determined for Extreme Event Limit State Load Combination I, except where the pier is designed as a column in its weak direction. If the pier is designed as a column, the design forces in the weak direction shall be as specified in Section 2.2.1.26 and all the design requirements for columns, as specified in Section 5 of AASHTO for concrete structures, shall apply. When the forces due to plastic hinging are used in the weak direction, the combination of forces, specified in Section 2.2.1.17, shall be applied to determine the elastic moment which is then reduced by the appropriate R-factor

Foundation Design Forces

The design forces for foundations including footings, pile caps and piles may be taken as either those forces determined for the Extreme Event Load Combination I, with the seismic loads combined as specified in Section 2.2.1.17, or the forces at the bottom of the columns corresponding to column plastic hinging as determined in Section 2.2.1.17. When the columns of a bent have a common footing, the final force distribution at the base of the columns in Step 4 of Section 2.2.1.26 may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other

2.3. ANALYSIS METHODS

2.3.1. BRIDGES

2.3.1.1. Australian codes (AS 5100 and AS 1170.4)

As per the Australian design standard, there are different methods of analysis specified depending on relevant additional requirements on bridge design category. Following table summarised all the required method of analysis according to the design category.

TABLE 2.16: 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	Horizontal and/or

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

Static analysis

Earthquake design force calculate based on Static analysis is described in C under loads and load combinations in Australian standards.

Dynamic analysis

As per AS 5100, dynamic analysis should be performed either with a response spectrum analysis or a time history analysis.

For the response spectrum analysis method in accordance with AS 1170.4, scaling of results, directional effects and torsion are not applicable to bridge structures, and can be ignored. A sufficient number of modes of free vibration should be included in the total response so that, for each direction, at least 90% of the structure's mass has been accounted for in the participating mass.

The effects of dynamic earthquake forces should be considered in the horizontal directions corresponding to the direction of each principal axis, or in the major orthogonal directions of the structure and the vertical direction. The effects in each direction shall be considered independently.

The analysis shall take account of torsional effects by use of a suitable three-dimensional mathematical model of the structure, which represents the spatial distribution of the mass and stiffness of the structure to an extent which is adequate for the determination of the significant features of its dynamic response.

2.3.1.2. Euro code (EC 8- Part 2)

Linear analysis with behaviour factor

The linear analysis using a global force reduction factor (behaviour factor q) is the normal analysis method. Response spectrum analysis may be applied in all cases, while equivalent static analysis with various simplifications is permitted under certain conditions. TABLE 2.17 gives the maximum values of the behaviour factor q. For reinforced concrete ductile members the values of q-factors specified in Table 4 are applicable when the normalised axial force η_k does not exceed 0.30. When $0.30 < \eta_k \leq 0.60$, even in a single ductile member, the value of the behaviour factor should be reduced to:

$$q_r = q - \frac{\eta_k - 0.3}{0.3}(q - 1) \geq 1.0 \quad (2.24)$$

$$\eta_k = {}^{N_{Ed}}/_{A_c f_{ck}} \tag{2.25}$$

N_{Ed} is the value of the axial force at the plastic hinge corresponding to the design seismic combination, positive if compressive, A_c is the area of the section and f_{ck} is the characteristic concrete strength.

The values of the q-factor for ductile behaviour specified in TABLE 2.17, may be used only if the locations of all the relevant plastic hinges are accessible for inspection and repair. Otherwise, these values are multiplied by 0,6; however final q-values less than 1.0 need not be used. When the main part of the design seismic action is resisted by elastomeric bearings the flexibility of the bearings imposes a practically elastic behaviour of the system. Such bridges are designed in accordance with the rules of seismic isolation. The inertial response of bridge structures whose mass follows essentially the horizontal seismic motion of the ground ("locked-in" structures), may be assessed using the design value of the seismic ground acceleration and q = 1. Abutments flexibly connected to the deck belong to this category.

TABLE 2.17: MAXIMUM VALUE OF THE BEHAVIOUR FACTOR Q

Type of ductile members	**Seismic behaviour**	
	Limited ductile	**Ductile**
Reinforced concrete piers		
Vertical piers in bending ($\alpha_s^* \geq 3.0$)	1.5	3.5 $\lambda(\alpha_s)$
Inclined struts in bending	1.2	2.1 $\lambda(\alpha_s)$
Steel piers		
Vertical piers in bending	1.5	3.5
Inclined struts in bending	1.2	2.0
Piers with normal bracing	1.5	2.5
Piers with eccentric bracing	-	3.5
Abutments rigidly connected to deck		
In general	1.5	1.5
Locked in structures (Par (9),(10))	1.0	1.0
Arches	1.2	2.0
$\alpha_s = {}^{L}/_{h}$ is the shear ratio of the pier, where L is the distance from the plastic hinge to the point if zero moment and h is the depth of the cross section in the direction of flexure of the plastic hinge. For $\alpha_s \geq 3,\ \lambda(\alpha_s)$ and $3 > \alpha_s \geq 1, \lambda(\alpha_s) = \sqrt{\frac{\alpha_s}{3}}$		

Regular and irregular seismic behaviour of ductile bridges Designating by $M_{Ed,i}$ the maximum value of design moment under the seismic load combinations at the intended location of plastic hinge of ductile member i, and by $M_{Rd,i}$ the design flexural resistance of the same section, with its actual reinforcement, under the concurrent action of the other action effects of the seismic load combination (Eq. (2.34)), then the required local force reduction factor r_i associated with member i, under the specific seismic action is

$$r_i = qM_{Ed,i} / M_{Rd,I} \tag{2.26}$$

One or more ductile members (piers) may be exempted from the above calculation of r_{min} and r_{max}, if the sum of their shear contributions does not exceed 20% of the total seismic shear in the direction under consideration. Bridges not meeting condition for regular bridges, shall be considered to have irregular seismic behaviour, in the direction under consideration. Such bridges should either be designed using a reduced q-value:

$$q_r = q\rho_o/\rho_r \geq 1.0 \tag{2.27}$$

Combination of modal responses and of the components of seismic action either the SRSS or the complete CQC modal combination rules are applicable. The design seismic action effects A_{Ed} should be derived from the most adverse of the following combinations:

$$A_{Ex} + 0.30A_{Ey} + 0.30\,A_{Ez} \tag{2.28}$$

$$0.30A_{Ex} + A_{Ey} + 0.30A_{Ez} \tag{2.29}$$

$$0.30A_{Ex} + 0.30A_{Ey} + A_{Ez} \tag{2.30}$$

A_{Ex}, A_{Ey} and A_{Ez} are the seismic actions in each direction X, Y and Z respectively.

Non - linear dynamic time-history analysis

In general, this method is used in combination with a normal response spectrum analysis to provide insight into the post - elastic response and comparison between required and available local ductilities. Generally, the results of the non-linear analysis are not intended to be used to relax requirements resulting from the response spectrum analysis. However, in the case of bridges with isolating devices and irregular bridges, lower results from a rigorous time-history analysis may be substituted for the results of the response spectrum analysis.

Static non-linear analysis (pushover analysis)

Pushover analysis is a static non-linear analysis of the structure under constant vertical (gravity) loads and monotonically increased horizontal loads, representing the effect of an horizontal seismic component. Second order effects should be accounted for. The horizontal loads are increased until the target displacement is reached at the reference point. This analysis should be used (alternatively to non - linear dynamic time-history analysis) in the case of irregular bridges.

- Analysis directions, target displacements and reference point

The analysis should be carried out in the following two horizontal directions; the longitudinal direction x, as defined by the centres of the two end-sections of the deck and the transverse direction y, that should be assumed at right angles to the longitudinal direction. The target displacement is the maximum of the displacements in the relevant direction, at the centre of mass of the deformed deck, resulting from equivalent linear multi-mode spectrum analysis, assuming q = 1.0, for the following combinations of seismic components: E_x "+" $0.3E_y$ and E_y "+" $0.3E_x$. The spectrum analysis should be carried out using effective stiffness of ductile members. The reference point should be the centre of mass of the deformed deck.

Load distribution The horizontal load increments $\Delta F_{i,j}$ assumed acting on lumped mass G_i/g, in the direction investigated, at each loading step j, are taken equal to:

$$\Delta F_{i,j} = \Delta\alpha_j G_i \zeta_i \qquad (2.31)$$

$\Delta\alpha_j$ is the horizontal load increment, normalized to the weight G_i, applied in step j, and ζ_i is a shape factor defining the load distribution along the structure, as follows.

a) constant along the deck, where
for the deck $\zeta_i = 1$ and for the piers connected to the deck

$$\zeta_i = z_i / z_P \qquad (2.32)$$

z_i is the height of point i above the foundation of the individual pier and
z_P is the height of the pier P (distance from the ground to the centreline of the deck) b) proportional to the first mode shape, where ζ_i is proportional to the component, in the direction investigated, of the modal displacement at point i, of the first mode, in the same direction. The mode having the largest participation factor in the direction under investigation should be considered as first mode in this direction.

Capacity design of members

For structures of ductile behaviour, capacity design effects FC are calculated by analysing the intended plastic mechanism under the permanent actions and the level of seismic action at which all intended flexural hinges have developed bending moments equal to an appropriate upper fractile of their flexural resistance, called the over strength moment M_o. This calculation should be carried out on the basis of equilibrium conditions, while reasonable approximations regarding the compatibility of deformations are acceptable.

The capacity design effects need not be taken greater than those resulting from the design seismic combination where the design effects A_{Ed} are multiplied by the q factor used. The over strength moment of a section is calculated as:

$M_o = \gamma_0 M_{Rd}$ (2.33)

γ_0 is the over strength factor M_{Rd} is the design flexural strength of the section, in the selected direction and sense, based on the actual section geometry, including reinforcement where relevant, and material properties (with γM values for fundamental load combinations). In determining M_{Rd}, biaxial bending under the permanent effects, and the seismic effects corresponding to the design seismic action in the selected direction and sense, shall be considered. The value of the over strength factor should reflect the probable deviation of material strength, and strain hardening. Recommended values are: Concrete members:

$\gamma_0 =$ 1.35(1+2(ηk-0,1)2) for confined sections with ηk > 0.1
$\gamma_0 =$ 1.35 for other concrete members Steel members:
$\gamma_0 =$ 1.25 Within members containing plastic hinge(s),

The capacity design bending moment M_c at the vicinity of the hinge shall not be assumed greater than the relevant design flexural resistance M_{Rd} of the hinge assessed.

Design seismic combination

The design value of action effects E_d, in the seismic design situation, are derived from the following combination of actions:

$G_k + P_k + A_{Ed} + \psi_{21} Q_{1k} + Q_2$ (2.34)

G_k are the permanent loads with their characteristic values, P_k is the characteristic value of prestressing after all losses, A_{Ed} is the most unfavourable combination of the components of the earthquake action in accordance with Eq 2.28-2.30, Q_{1k} is the characteristic value of the traffic load, and ψ_{21} is the combination factor with recommended values ψ_{21}=0 in general, ψ_{21} = 0.2 for road bridges with intense traffic and ψ_{21} = 0.3 for railway bridges. Q_2 is the quasi permanent value of actions of long duration (e.g. earth pressure, buoyancy, currents etc.) Actions of long duration are considered to be concurrent with the design earthquake. Seismic action effects need not be combined with action effects due to imposed deformations (temperature variation, shrinkage, settlements of supports, ground residual movements due to seismic faulting)

US design guidelines

Minimum analysis requirements for seismic effects as per AASHTO for seismic analysis for bridges are given in following table.

TABLE 2.18: MINIMUM ANALYSIS REQUIREMENTS FOR SEISMIC EFFECTS

Seismic Zone	Single-Span Bridges	Multi-span Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		regular	irregular	regular	irregular	regular	irregular

1	No seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

* = no seismic analysis required

UL = uniform load elastic method

SM = single-mode elastic method

MM = multimode elastic method

TH = time history method

Except as specified below, bridges satisfying the requirements of TABLE 2.18 may be taken as "regular" bridges. Bridges not satisfying the requirements of TABLE 2.19 shall be taken as "irregular" bridges.

TABLE 2.19: REGULAR BRIDGE REQUIREMENTS

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutments	-	4	4	3	2

Curved bridges comprised of multiple simple-spans shall be considered to be "irregular" if the subtended angle in plan is greater than 20 degrees. Such bridges shall be analyzed by either the multimode elastic method or the time-history method.

Single-Mode Methods of Analysis

There are two methods described under single mode method of analysis.

- Single-Mode Spectral Method
- Uniform Load Method

Single-Mode Spectral Method

The single-mode method of spectral analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. For regular bridges, the fundamental modes of vibration in the horizontal plane coincide with the longitudinal and transverse axes of the bridge structure. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, C_{sm}, specified in Eq. (2.17) , and the corresponding spectral displacement. This amplitude shall be used to determine force effects

Uniform Load Method

The uniform load method shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction of the base structure. The period of this mode of vibration shall be taken as that of an equivalent single mass-spring oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. The elastic seismic response coefficient, Csm, Eq. (2.17) shall be used to calculate the equivalent uniform seismic load from which seismic force effects are found.

Multimode Spectral Method

The multimode spectral analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure. The number of modes included in the analysis should be at least three times the number of spans in the model. The design seismic response spectrum as specified in Eq. (2.17) shall be used for each mode. The member forces and displacements may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (CQC) method.

Time-History Method

Developed time histories shall have characteristics that are representative of the seismic environment of the site and the local site conditions. Response-spectrum-compatible time histories shall be used as developed from representative recorded motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching.

Where recorded time histories are used, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. Each time history shall be modified to be response-spectrum compatible using the time-domain procedure.

At least three response-spectrum-compatible time histories shall be used for each component of motion in representing the design earthquake (ground motions having seven percent probability of exceedance in 75 yr). All three orthogonal components (x, y, and z) of design motion shall be input simultaneously when conducting a nonlinear time-history analysis. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction.

If a minimum of seven time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction. For near-field sites, the recorded horizontal components of motion that are selected should represent a near-field condition and should be transformed into principal components before making them response-spectrum-compatible. The major principal component should then be used to represent motion in the fault-normal direction and the minor principal component should be used to represent motion in the fault-parallel direction.

2.4.SPECIAL DETAILING PROCEDURE

2.4.1. Steel Bridges

2.4.1.1. Australian codes (AS 5100 and AS 1170.4)

For all bridges, good detailing practices and design for ductile behaviour shall be employed where practicable, to guard against the effects of unexpected seismic disturbances. Sufficient ductility to deal with unexpected seismic disturbances shall be deemed to be achieved in bridges with a Bridge Design Category of BEDC-1 or BEDC-2 if the structure is analysed using a response factor (R_f) equal to 2.0, and the elements designed for the resulting actions. Particular attention shall be given to the prevention of dislodgment of the superstructure from its support system and the provision of viable, continuous and direct load paths from the level of the bridge deck to the foundation system

For bridge structures in BEDC-2, BEDC-3 and BEDC-4, a clearly defined collapse mechanism shall be established. The structural members shall be ductile at the potential plastic hinge locations defined in the mechanism.

Minimum ductility requirements for the design of these structural members under earthquake design loads shall be as specified in AS 5100.5 and AS 5100.6. These requirements are to ensure that the required ductility at potential plastic hinges can be achieved.

Further AS 5100.2 Clause 14.4.3 to 5 states "detailing of structural members, restraining devices, bearing and deck joints shall conform to clause 14.7"

2.4.1.2. Euro code (EC 8- Part 2)

In bridges designed for ductile behaviour, the design values of the axial force, and shear forces, $V_{E,d}$, in piers consisting of moment resisting frames shall be assumed to be equal to the capacity design action effects I_{Ye} and V_c, respectively. The design values for the axial shear force shall be taking the force in all diagonals as corresponding to the over strength of the weakest diagonal.

In bridges designed for ductile behaviour ($q > 1.5$) the deck shall be verified for the capacity design effects. In bridges designed for limited ductile behaviour the verification of the deck shall be carried out using the design action effects from the analysis.

Hollow piers

Unless appropriate justification is provided, the ratio b/h of the clear width "b" to the thickness "h" of the walls, in the plastic hinge region of hollow piers with a single or multiple box cross-section, should not exceed 8.

(3) For hollow cylindrical piers, above limitation applies to the ratio D_i /h, where D_i is the inside diameter.

(4) In piers with simple or multiple box section and when the value of the ratio η_k does not exceed 0.20, there is no need for verification of the confining reinforcement provided that the requiren1ents of 6.2.2 of EC 8 [9] are met.

Non-ductile structural components, such as fixed bearings, sockets and anchorages for cables and stays and other non-ductile connections shall be designed using either seismic action effects multiplied by the q-factor used in the analysis, or capacity design effects. The latter shall be determined from the strength of the relevant ductile members (e.g. the cables) and an overstrength factor of at least 1.3.

2.4.1.3. US design guidelines

Structural steels used within the seismic load path shall meet the requirements of AASHTO [3] Section 6.4.1, except as modified herein.

For steel-girder bridges located in Seismic Zone 1, the design of all support cross-frame or diaphragm members and their connections and the connections of the superstructure to the substructure shall satisfy the minimum requirements specified in AASHTO [3] 3.10.9 and 4.7.4.4.

Components of slab-on-steel girder bridges located in Seismic Zones 3 or 4, shall be designed using one of the two types of response strategies. One of the two types of response strategies should be considered for bridges located in Seismic Zone 2:

Type 1—Design an elastic superstructure with a ductile substructure according to the provisions of Section 6.16.4.4 of AASHTO [3].

Type 2—Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and substructure according to the provisions of Section 6.16.4.4 of AASHTO [3]. The deck and shear connectors on bridges located in Seismic Zones 3 or 4 shall also satisfy the provisions of Section 6.16.4.2 and 6.16.4.3 of AASHTO [3] , respectively. Support cross-frame members on bridges located in Seismic Zones 3 or 4 shall be considered primary members for seismic design. Structural analysis for seismic loads shall consider the relative stiffness of the concrete deck, girders, support cross-frames or diaphragms, and the substructure.

Reinforced concrete decks attached by shear connectors satisfying the requirements of Section 6.16.4.3 of AASHTO [3] shall be designed to provide horizontal diaphragm action to transfer seismic forces to the supports as specified in this Article. Where the deck has a span-to-width ratio of 3.0 or less and the net mid-span lateral seismic displacement of the superstructure is less than twice the average of the adjacent lateral seismic support displacements, the deck within that span may be assumed to act as a rigid horizontal diaphragm designed to resist only the shear resulting from the seismic forces.

Otherwise, the deck shall be assumed to act as a flexible horizontal diaphragm designed to resist shear and bending, as applicable, resulting from the seismic forces.

Shear Connectors

Stud shear connectors shall be provided along the interface between the deck and the steel girders, or along the interface between the deck and the top of the support. cross-frames or diaphragms, or both, as necessary to transfer the seismic forces. If reinforced concrete diaphragms that are connected integrally with the bridge deck are used at support locations, then the shear connectors on the steel girders at those locations need not be designed according to the provisions of this Article. The shear connectors on the girders assumed effective at the support under consideration shall be taken as those spaced no further than 9tw on each side of the outer projecting element of the bearing stiffeners at that support. The diameter of the shear connectors within this region shall not be greater than 2.5 times the thickness of the top chord of the cross-frame or the top flange of the diaphragm. At support locations, shear connectors on the girders or on the support cross-frames or diaphragms, or both, as necessary, shall be designed to resist the combination of shear and axial forces corresponding to the transverse seismic shear force.

2.4.2. Concrete bridges

2.4.2.1. Australian code requirements (AS 5100 and AS 1170.4)

There are some special reinforcements detailing requirements in AS 5100.5 for concrete bridges. For reinforced concrete members, the area of tensile and compression reinforcement should be equal at sections where a plastic hinge is expected to develop. In addition, the member ultimate design axial compression force, under permanent loads and earthquake effects, at plastic hinge locations shall not be greater than 35% of the ultimate axial compression force capacity of the section.

For prestressed concrete members, in plastic hinge regions at least 40% of the total tensile steel shall be non-prestressed reinforcement.

The flexural strength shall be greater than 1.3 times the cracking moment at that section, after allowance for the effect of axial loads

For bridge structures in BEDC-2, BEDC-3 and BEDC-4, special consideration shall be given to the detailing of concrete compression members, bearing in mind the manner in which earthquake-induced energy will be dissipated and the desirability of avoiding brittle failures, especially in shear. In particular, the ultimate shear capacity shall be assessed and additional capacity provided, where necessary, to ensure that premature failure does not occur. NOTE: The Clause does not apply to bridge structures in BEDC-1.

In reinforced and prestressed concrete compression members, the longitudinal reinforcement shall be restrained by lateral reinforcement in the potential plastic hinge regions as follows:

(a) Where helices are used, the area of the reinforcement in the helix, per unit length of member is given as,

$$\frac{A_s}{s} \geq \frac{0.03 f_c' D_c}{f_{sy,f}} \qquad (2.35)$$

(b) Where closed ties are used, the total cross-sectional area of the ties (Asv), including supplementary cross-ties, should not be less than

$$0.30 s y_1 \left[\left(\frac{A_g}{A_c}\right) - 1\right]\left[\frac{f_c'}{f_{sy,f}}\right] \text{ or} \qquad (2.36)$$

$$0.09 s y_1 \left[\frac{f_c'}{f_{sy,f}}\right] \qquad (2.37)$$

Whichever is greater

where

A_s = area of the reinforcement forming the helix

s = centre-to-centre spacing of ties

f_c'= characteristic compressive cylinder strength of concrete at 28 days

D_c = diameter of the inside face of the helix

$f_{sy,f}$ = yield strength of the reinforcement used as fitments

y_1 = larger core dimension of a closed rectangular tie

A_g = area of the gross cross-section of the member

A_c = area of the cross-section of the core measured over the outside of the ties

except that Item (b)(i) does not apply if φN_{uo} for the core (concrete + reinforcement) is greater than N*.

(c) Closed ties in accordance with Item (b) shall be used singly or in sets spaced at not more than 150 mm centres, or one-quarter of the minimum cross-section dimension, whichever is smaller.

Supplementary ties, of the same diameter as the closed ties, consisting of a straight bar with a 135° minimum hook at each end, may be considered as part of a closed tie if they are spaced at not more than 350 mm centres and secured with the closed tie to the longitudinal bars.

(d) The lateral reinforcement in accordance with Item (a) or Item (b) shall extend into the footing, pile cap or deck, as applicable, over a length not less than half the maximum dimension of the compression member or 400 mm, whichever is greater

The lateral reinforcement shall extend for a minimum distance of twice the maximum dimension of the compression member from the top and bottom of framed piers, or from the bottom of cantilever piers.

Piles may have potential plastic hinge positions at the top of the piles and at locations down the pile where there is an abrupt change in soil stiffness. The lateral reinforcement shall extend for a minimum distance of twice the maximum dimension of the pile from the bottom of the pile cap, or four times the maximum pile dimension centred about the hinge location.

Restraining devices

Where the horizontal restraints of conventional bearings are inadequate under earthquake effects, restraining devices, such as ties, shear keys, stops and dowels, shall be provided with the specific aim of preventing dislodgment of the superstructure from the support structure. Restraining devices and connections shall be designed to withstand the horizontal design earthquake forces. Vertical restraint devices shall be provided at all supports where the vertical design earthquake force opposes and is greater than 50% of the static reaction under permanent loads. The vertical restraint device shall be designed to resist not less than 10% of the vertical reaction from the permanent effects of the support.

Due to the nature of earthquake loads, horizontal restraints cannot be assumed to rely on any component of friction. For assessment of the structure under any load combination which includes earthquake effects, the friction coefficient between any material types shall be equal to zero.

Bearings and deck joints shall accommodate the horizontal movements due to earthquake effects.

Where excessive movements, which are outside the range of conventional bearings or deck joints are expected, additional devices may be used to limit movements under earthquake loadings only. These special devices, such as buffer bearings, shall be designed to be activated after a large, but tolerable, horizontal movement to prevent failure of sliding bearings and deck joints.

Bearing seats supporting expansion ends of the superstructure for bridges in BEDC-2, BEDC-3 and BEDC-4 shall be designed to provide a minimum support length measured normal to the face of an abutment or pier (L_{bs}) of

$$L_{bs} = (200 + 1.7L_d + 6.7h_d)(1 + 0.000125\theta_s^2) * 10^{-3} \tag{2.38}$$

where

L_d = length of the superstructure to the next expansion joint

h_d = average height of the columns or piers supporting the superstructure length Ld

θ_s = angle of skew of the support measured from a line normal to the span

For bridge structures in BEDC-2, BEDC-3 and BEDC-4, the connection between each pile and its pile cap shall be designed to resist a tensile force of not less than 10% of N* for the pile.

2.4.2.2. Euro code requirements (EC 8- Part 2)

Detailing is a very important feature in design of bridges with concrete. In this Section, some important aspects are discussed in detail.

Confinement

Confinement In potential hinge regions where the normalised axial force exceeds the limit: $\eta_k = N_{Ed}/A_c f_{ck} > 0{,}08$, confinement of the compression zone is in general necessary. No confinement is required in piers with flanged sections (box- or I-Section) if, under ultimate seismic load conditions, a curvature ductility $\mu_\Phi = 13$ for bridges of ductile behaviour, or $\mu_\Phi = 7$ for bridges of limited ductile behaviour, is attainable with the maximum compressive strain in the concrete not exceeding the value of $\varepsilon_{cu} = 0{,}35\%$. In cases of deep compression zones, the confinement may be limited to that depth in which the compressive strain exceeds $0{,}5\varepsilon cu$ The quantity of confining reinforcement is defined by the mechanical reinforcement ratio: $\omega_{wd} = \rho_w.f_{yd}/f_c$ where, ρw is the transverse reinforcement ratio equal to

for rectangular sections,

$$\rho_w = A_{sw}/s_L b \tag{2.39}$$

and for circular sections,

$$\rho_w = 4A_{sp}/D_{sp}.s_L \tag{2.40}$$

The minimum amount of confining reinforcement shall be determined as follows: a) for rectangular hoops and cross-ties, in each direction.

$$\omega_{wd,r} \geq \max\left(\omega_{w,req}, \frac{2}{3}\omega_{\mathrm{w,min}}\right) \tag{2.41}$$

$$\omega_{w,req} = \frac{A_c}{A_{cc}}\lambda\eta_k + 0.13\frac{f_{yd}}{f_{cd}}(\rho_L - 0.01) \tag{2.42}$$

A_c is the gross concrete area of the section, A_{cc} is the confined (core) concrete area of the section, λ factor specified in TABLE 2.20 and ρ_L is the reinforcement ratio of the longitudinal reinforcement.

TABLE 2.20: MINIMUM VALUES OF Λ AND Ω$_{W,MIN}$

Seismic Behaviour	**λ**	**$\omega_{w,min}$**
Ductile	0.37	0.18
Limited Ductile	0.28	0.12

Buckling of longitudinal compression reinforcement

Buckling of longitudinal reinforcement shall be avoided along potential hinge areas even after several cycles into the plastic region. Therefore all main longitudinal bars shall be restrained against outward buckling by transverse reinforcement (hoops or cross-ties) perpendicular to the longitudinal bars at a maximum (longitudinal) spacing $s_L = \delta_{\phi L}$, where ϕ_L is the diameter of the longitudinal bars.

Minimum overlap lengths

At supports where relative displacement between supported and supporting members is intended under seismic conditions, a minimum overlap length shall be provided. This overlap length shall be such as to ensure that the function of the support is maintained under extreme seismic displacements. At an end support on an abutment and in the absence of a more accurate estimation the minimum overlap length l_{ov} may be estimated as follows:

$$l_{ov} = l_m + d_{eg} + d_{es} \quad (2.43)$$

$$d_{eg} = \varepsilon_s L_{eff} \leq 2d_g \quad (2.44)$$

$$\varepsilon_s = \frac{2d_g}{L_g} \quad (2.45)$$

l_m is the minimum support length securing the safe transmission of the vertical reaction ≥ 40cm,

d_{eg} is the effective displacement of the two parts due to differential seismic ground displacement,

d_g is the design value of the peak ground displacement = 0.025agSTCTD

L_g is the Characteristic distance as shown below

TABLE 2.21: DISTANCE BEYOND WHICH GROUND MOTIONS MAY BE CONSIDERED UNCORRELATED

Ground Type	A	B	C	D	E
L_g(M)	600	500	400	300	500

Other rules for reinforcement detailing

Due to the potential loss of concrete cover in the plastic hinge region, the anchorage of the confining reinforcement shall be effected through 135° hooks surrounding a longitudinal bar plus adequate extension (min. 10 diameters) into the core concrete. Similar anchoring or full strength weld is required for the lapping of spirals or hoops within potential plastic hinge regions. In this case laps of successive spirals or hoops, when located along the perimeter of the member, should be displaced in accordance with 8.7.2 of EN1992-1. No splicing by lapping or welding of longitudinal reinforcement is allowed within the plastic hinge region.

2.4.2.3. US design guideline requirements

Transverse reinforcement shall be anchored at both ends. For composite flexural members, extension of beam shear reinforcement into the deck slab may be considered when determining if the development and anchorage provisions.

The design yield strength of non-prestressed transverse reinforcement shall be taken equal to the specified yield strength when the latter does not exceed 60.0 ksi. For non-prestressed transverse reinforcement with yield strength in excess of 60.0 ksi, the design yield strength shall be taken as the stress corresponding to a strain of 0.0035, but not to exceed 75.0 ksi. The design yield strength of prestressed transverse reinforcement shall be taken as the effective stress, after allowance for all prestress losses, plus 60.0 ksi, but not greater than f_{py}.

When welded wire reinforcement is used as transverse reinforcement, it shall be anchored at both ends. No welded joints other than those required for anchorage shall be permitted. Components of inclined flexural compression and/or flexural tension in variable depth members shall be considered when calculating shear resistance.

Lap Splices in Tension

The length of lap for tension lap splices shall not be less than either 12.0 in. or the following for Class A, B or C splices:

Class A splice 1.0 ℓ_d

Class B splice 1.3 ℓ_d

Class C splice 1.7 ℓ_d

The tension development length, ℓ_d, for the specified yield strength shall be taken in accordance with Article 5.11.2. The class of lap splice required for deformed bars and deformed wire in tension shall be as specified in following Table.

TABLE 2.22: REQUIRED LAP LENGTH BASED ON AREA OF REINFORCEMENT

Ratio of (A_s provided) /(A_s required)	Percent of As Spliced with Required Lap Length		
	50	75	100
≥2	A	A	B
<2	B	C	C

Mechanical connections or welded tension splices, used where the area of reinforcement provided is less than twice that required, shall meet the requirements for full-mechanical connections or full-welded splices.

Mechanical connections or welded splices, used where the area of reinforcement provided is at least twice that required by analysis and where the splices are staggered at least 24.0 in., may be designed to develop not less than either twice the tensile force effect in the bar at the section or half the minimum specified yield strength of the reinforcement.

Splices of reinforcement in tension tie members shall be made only with either full-welded splices or full-mechanical connections. Splices in adjacent bars shall be staggered not less than 30.0 in. apart.

2.5. CASE STUDY

After reviewing the 3 bridge design standards, it is evident that AS 5100 [5]which is the Australian standard for the design of bridges is more relevant for the Australian conditions. Therefore in the case study analysis of a typical bridge in Australia, the design guidelines given in AS 5100 was utilised. Although AS 5100 was not the code of practice during the construction of this bridge, it was selected to identify the vulnerability and performance of the bridge due to possible earthquake loads in Australia.

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 [10]. 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 following Figures.

FIGURE 2.5: PHOTOS OF THE TENTHILL CREEK BRIDGE

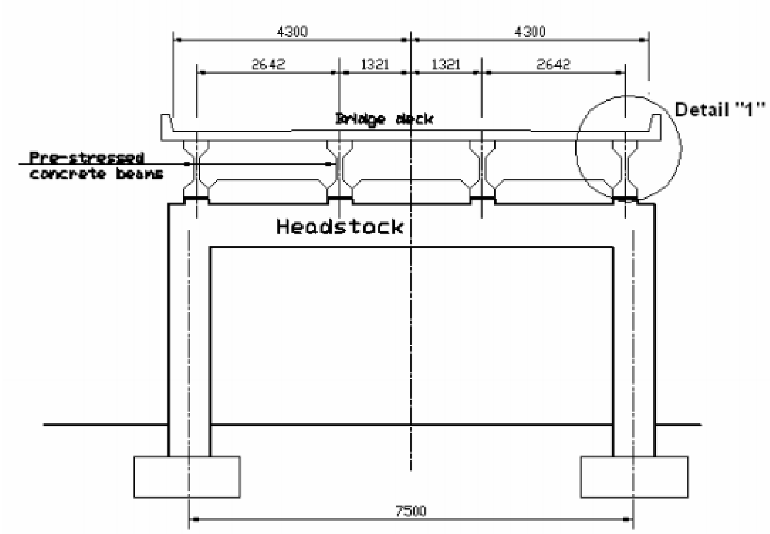


FIGURE 2.6: SECTION DETAIL OF THE TENTHILL CREEK BRIDGE [10]

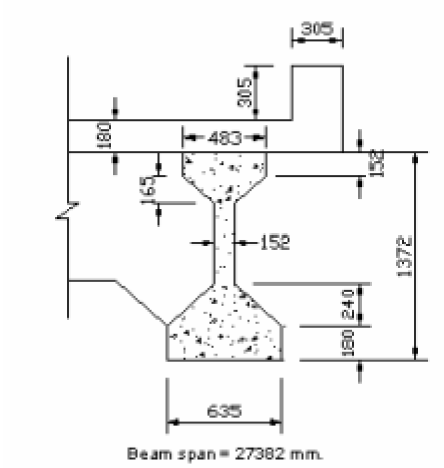


FIGURE 2.7: SECTION DETAIL OF PRE-CAST CONCRETE BEAM [10]

Design parameters

The bridge category is assumed to be BEDC (iV) accordance with AS 5100

The hazard factor is 3.68 in accordance with AS 1170.4

The site sub soil class is assumed to be shallow soil (C_e) as per AS 1170.4

Total dead load of the bridge is calculated as 11951kN.

Important factor 1.25

C=0.166

S=3.68

Rf=4

Gg =11951kN

Design assumptions

- The bridge abutments and foundations are fully rigid.
- The pier shear deformation is ignored

Obtaining all the required inputs and applying them in equation 2.1, the horizontal static earthquake force becomes 2288kN.

As per the code AS 5100, dynamic analysis should be carried out for this structure. Therefore the response spectrum analysis using ANSYS was carried out with spectral coordinates given in AS 1170.4. The stress distribution of the bridge due to the response spectrum analysis is shown in FIGURE 2.8 with the maximum stress of 252 MPa at column base in each column.

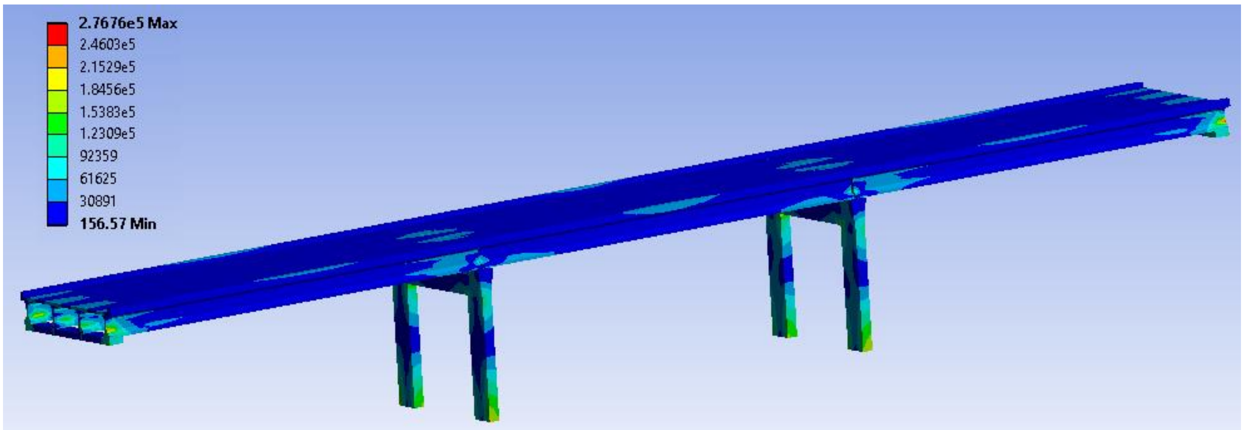


FIGURE 2.8: STRESS DISTRIBUTION FOR RESPONSE SPECTRUM ANALYSIS

2.6. DISCUSSION AND RECOMMENDATIONS

All the above noted codes imply that bridge designed according to each code provisions satisfy a minimum level of performance. The difference between the various codes is that degree to which these performance criteria are explicitly defined and checked in design process. The common concept for all three codes is the concept of acceptable damage provided that collapse of all the bridge does not occur even in higher earthquakes with strong shaking. As per AASHTO specifications, acceptable damage is defined as mean flexural yielding in the columns only (No shear failure) and even it must be detectable and repairable. All other damage to foundations, abutments, shear keys, connection etc) is unacceptable. This definition is generally used by all codes. Therefore in all the codes, it is assumed that bridges are designed without any damage and satisfy the requirements for more frequent smaller earthquakes.

There is only one level design approach for most of the ordinary bridges other than important bridge aspects in most of the codes. This occurs due to the assumption that if one level of performance is satisfied for ordinary bridges, then next level is performance is automatically satisfied by default. However sometimes it is essential to carry out simplified design procedures with standardize checkpoints to satisfy the criteria [1].

Current elastic analysis methods given in all the codes are generally static analysis and response spectrum methods are generally valuable analysis methods. However the range of applicable structures may be further refined and simplified to obtain better representation of the bridges.

Use of structural response factor or modification factor is an important feature in all the codes. This factor is used to obtain the design forces from elastic analysis and will remain a key step in seismic design. Therefore some improvements and estimation of this factor for different types of structures with different parts are essential [1]

Steel bridges are not well advanced compared to concrete bridges and need more improvements such as detailing of sections and joints for the designed ductility levels. As per Clause 13.4 of As 4100, there are no special detailing guidelines provided for steel bridge structures. Further some codes have utilised displacement based methods or energy based method in seismic design. Therefore these types of acceptable alternative methods with performance based of bridges are essentials.

The seismic design provisions in the Australian bridge code AS 5100 have recently been revised to include several improvements and remove ambiguities such as application of plastic hinge detailing and response factor magnitudes for different bridge configurations found in the 2004 edition of the code. DR AS 5100.2: 2014 provides two design approaches, namely the displacement-based method (DBM) and the force based method (FBM). The latter method uses generalised acceleration spectra to determine the seismic design actions, while the former uses equivalent displacement spectra. The FBM provides a conventional approach that can be easily applied using one of the commercially available software products. The DBM enables designers to predict the bridge behaviour at design seismic events, and determine the required ductile detailing accordingly. The DBM provides a tool to check whether a bridge will remain elastic at design seismic events and can be exempt from excessive seismic ductile detailing. However in this report the draft AS 5100: 2014 was not reviewed. But it will be reviewed in future work in this project.

3. FLOOD LOADING ON ROAD STRUCTURES

3.1. DESIGN STANDARDS ANALYSIS

3.1.1. BRIDGES

3.1.1.1. AS 5100

Limit state

AS 5100 [11] states that "*The ultimate limit states define the capability of a bridge to withstand, without collapse, any flood of a magnitude up to and including that with a 2000 year average return interval, whichever produces the most severe effect. It can be accepted that scour of the stream bed and considerable damage to approaches and embankments may take place, provided that the structural integrity of the bridge is maintained.*"

"*As the critical design condition may occur at the flood level which just causes overtopping of the superstructure, an estimate of the return interval of such a flood shall be made and, if appropriate, this condition shall be considered in the design. Where the critical design condition occurs at an average return interval of less than* ***2000 years****, the ultimate load factor (γ_{WF}) shall be obtained from the following figure, but shall be not greater than 2.0.* "[11]

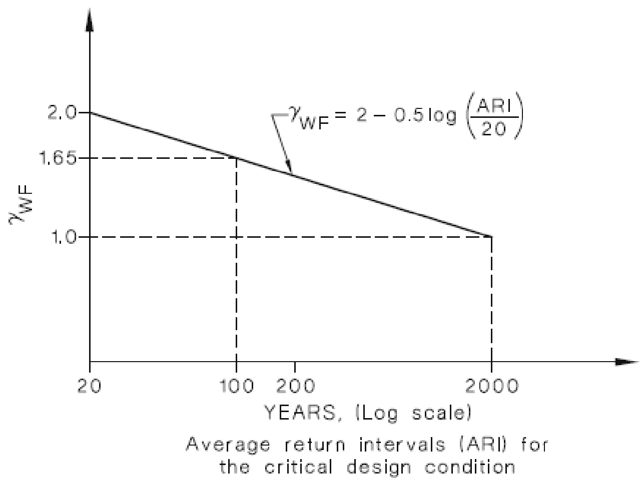


FIGURE 15.2.1 ULTIMATE LOAD FACTOR (γ_{WF})

Serviceability limit states

The serviceability limit states define the capability of the road and bridge systems to remain open during a serviceability design flood or to sustain an overtopping flood without damage to bridges, culverts, floodways or embankments within the system. The serviceability design flood shall be that with a ***20 year*** *average return interval.*

3.1.1.1. American standards

AASHTO [12] states that the extreme event limit state shall be taken to ensure the structural survival of a bridge during a major flood, or when collided by a vessel, vehicle, or ice flow, possibly under scoured conditions.

ASCE [13] asserts that "*the design flood should at least be equivalent to the flood having a 1 percent chance of being equaled or exceeded in any given year (i.e., the base flood or* ***100-year*** *flood, which served as the load basis in ASCE 7-95). In some instances, the design flood may exceed the base flood in elevation or spatial extent; this excess will occur where a community has designated a greater flood (lower frequency, higher return period) as the flood to which the community will regulate new construction.*"

3.1.2. FLOOD-WAYS

Floodway design process is mainly governed by the hydraulic conditions and associated properties of the creek bed in the locality. Austroads publication, "Guide to Road Design – Part 5B: Drainage – Open Channels, Culverts and Floodways" [14] outlines basic floodway design aspects and serves as the national floodway design guideline. However, road authorities in different regions across Australia rely on their own floodway design guidelines mainly due to the variation in hydraulic conditions, material availability and traditional practices of the locality. This section of the report evaluates Australian floodway design guidelines as well as available international floodway design guidelines.

Sections 3.1.2.1 – 3.1.2.6 review important design considerations for floodways in Australia. These design considerations are based on below three Australian floodway design guidelines.

1. Austroads publication: Guide to Road Design Part 5: Drainage [14],
2. Queensland Transport and Main Roads publication: Road Drainage Manual [15], and
3. Main Roads Western Australia Publication: Floodway Design Guide [16].

Specific references for these three sources are provided only to highlight unique information or differences between guidelines.

3.1.2.1. Definition for Floodways in Australian Guidelines

Floodways are defined almost in a unique way in all three Australian Floodway Design Guidelines highlighting the fact that floodways are mainly designed based on the hydraulic conditions. These definitions are repeated below.

Definition 1 (Guide to Road Design Part 5: Drainage [14]): A length of pavement on a typically level grade that is designed to be overtopped by floodwater during relatively low average recurrence interval (ARI).

Definition 2 (Chapter 10 of the Road Drainage Manual [15]): Sections of roads which have been designed to be overtopped by floodwater during relatively low average recurrence interval (ARI) floods.

Definition 3 (The Floodway Design Guide [16]): A roadway across a shallow depression subject to flooding, specifically designed to overtop and constructed to resist the damaging effects of overtopping. This guideline also mentions that floodway is a special case of causeway. A causeway is defined as "a roadway across a watercourse or across tidal water, specifically designed to resist submergence".

3.1.2.2. Floodway analysis and design process

Existing floodway design process is based on hydraulic aspects. This requires hydrological investigation and hydraulic analysis in conjunction with serviceability levels of the floodway. Also, designer should pay attention to other consideration such as environmental factors. This is further discussed in section 3.3.2.

3.1.2.3. Floodway serviceability

Floodway serviceable level is defined based on the safer flood level for crossing vehicles. In general it is accepted that higher serviceability levels result in increased cost of the structure. Therefore, floodways are designed to be submerged and closed for traffic for defined flood events. When a crossing is to be designed for overtopping (similar situation as floodways), it is important to know frequency and duration of the time period that it will be submerged. On the other hand, the designer should take into account the expected level of service by considering community expectations, availability of alternative routes, anecdotal and historical information of road closure and damage, importance of road to access emergency and post-disaster recovery situations and the relationship between traffic density and composition [16].

Time of Submergence (ToS)

Time of submergence (ToS) which is a road design paprameter, indicates the duration that the road is inundated [14]. Higher TOS indicates that floodway has potential to submerge frequently for short period of times or less frequently for long period of time. Both of these situations can cause detrimental effect to the floodway causing stability issues in embankments and pavements leading to increased maintenance costs. TOS is expressed in two main ways: time of submergence during a major flood and Average Annual Time Of Submergence (AATOS). The time of submergence during a major flood event is expressed in hours of submergence for a given flood event. AATOS represents the average time per year that the road is submerged and typically expressed in hours per year. TOS is useful to compare several crossings or upgrading options.

Although it is accepted that the higher TOS results with stability issues and higher maintenance costs, scientific judgements or a limiting TOS is not defined.

Time Of Closure (TOC)

Time of closure is closely related with the serviceability of the road segment and is used to calculate the expected delays to traffic. When the total head across the road exceeds a certain limit, it is recommended to close the road segment for traffic as outlined in the Table 23 below. The road drainage manual [15] adopts a constant limit of 300 mm total head to define the time of closure irrespective of the vehicle type. However, the floodway design guide [16] has outlined two values. Similar to TOS, TOC is also defined either in terms of time of closure during a major flood event or Average Annual Time Of Closure (AATOC).

Table 23: Maximum trafficable flood levels

	Guide to Road Design Part 5: Drainage [14]	Road Drainage Manual [15]	The Floodway Design Guide [16]
Maximum total head (depth plus velocity head) limit defining the instance of closure	300 mm	300 mm	for conventional cars: 300 mm (Critical depth of 200 mm) for heavy vehicles: 750 mm (critical depth of 500mm)
Critical depths based on Bonham and Hattersley (1967) Critical depth is two third of the total head	Not applicable	Not applicable	Ideal condition: 365 mm Defined limit: 230 mm (to account for presence of debris, potholes and waves etc...)

3.1.2.4. Geometric and safety considerations

The location and the associated geometric properties of a floodway are governed by safety of drivers during flood events. Floodway length, horizontal and vertical alignments are the three main geometric properties that need to be considered.

Length of a floodway is often limited to 300 m to allow sufficient time to recognize flood water over the road and to stop the vehicle at a safer distance. This helps to avoid disorientation of drivers. The floodway should be divided into shorter segments, if the proposed length is longer than 300 m. In such instances, raised road sections above the maximum flood level should be provided between two floodway sections. Table 24 summarises important geometric and safety features describes in the design guidelines.

TABLE 24: IMPORTANT GEOMETRIC AND SAFETY CONSIDERATIONS IN FLOODWAY DESIGN

	Guide to Road Design Part 5: Drainage [14]	Road Drainage Manual [15]	The Floodway Design Guide [16]
Length of the floodway	< 300m	< 300m	< 300m
Length > 300m	Break into shorter lengths (by providing sections of road that are above the maximum flood level).	Break into shorter lengths (by providing sections of road that are above the maximum flood level).	Care should be taken to avoid the creation of isolated islands.
Horizontal curve	Not recommended	Not recommended	Not recommended
Vertical curve	Not recommended to avoid variations in depths of flow	Not recommended to avoid variations in depths of flows	Use of sag curves to the approach ramps and skew crossing of a major stream are exempted. Crest curves should be designed to provide for adequate visibility
Embankment Cross-Section	Not discussed	Not discussed	Two-way crossfall is preferred One-way cross fall induces smooth, stable flow over the floodway, but may result in a hydraulic jump forming on the road surface One-way cross fall is used if the floodway must be constructed on a horizontal curve
Floodway signage	AS 1742.2		In accordance with the Austrlaian Standard 1742.2-1994, Manual of Uniform Traffic Control Devices-Part 2-Traffic Control

			Devices for General Use Required signs: "FLOODWAY": should be provided "ROAD SUBJECT TO FLOODING INDICATORS SHOW DEPTH": based on the depth of flooding Depth indicators Pair of guide posts approximately at every 25m to delineate the edge of the road pavement Use of Guard railing and other barriers are discouraged as they obstruct the flow over the floodway

3.1.2.5. Floodway protection structures

Scour Protection

Scour damage is one of the main damage types for floodways and hence the scour protection is a prime consideration. Six locations that are more vulnerable to scour damage and their causes are [16]:

1. Toe of the downstream batter slope – due to impinging super-critical velocity at the toe of the batter slope
2. Surface of batter slope – due to the drag/shear resistance on the batter slope
3. At the edge of downstream shoulders – due to an up lift force caused by the embankment geometry
4. On the road surface – due to shear/drag resistance on the running surface
5. On the upstream batter slope – Due to approach velocity
6. Scour below the floodway – Due to piping or riverbed instability caused by sediment transportation.

The floodway design guide [16] outlines seven possible protection techniques to overcome such scour damage:

1. Appropriately designed rock protection
2. Pump-up concrete revetment mattresses
3. Cut-off walls (end walls)
4. Rock fill below embankment
5. Cement stabilized batter slope / embankment fill
6. Cement stabilized subgrade / base course
7. Two-coat bituminous seal

The floodway design guide [16] also outlines that the scour protection requirements can be limited by adopting intelligent hydraulic design approaches. However, such solutions should be weighed against the serviceability requirements, site conditions and construction costs. For an example, the design of floodways to submerge at a low flow by lowering the floodway level or use of culverts to raise the tail-water level are possible solutions, in expense of serviceable level and cost of the floodway.

Pavement protection

Stabilized base course or concrete pavement are recommended pavements [16]. Selection of pavement type is based on the type and volume of traffic during the dry and wet conditions. Stabilized base course is recommended in areas where periods of inundation are relatively short. However, heavy traffic should not be allowed under submerged condition for stabilized base course pavements. Concrete pavements are generally recommended if the period of inundation is long.

Embankment batter protection

The embankment batter protections are mainly classified into flexible protection and rigid protection [16]. Dumped graded rock, hand placed graded rock, rock mattresses, flexible mats, flexible pump-up revetment mattresses and vegetative cover are outlined as flexible type batter protection methods. Grouted rock, rigid pump-up revetment mattresses and concrete slab are outlined under rigid batter protection methods. The selection of rigid protection methods should be carefully assessed by the design engineer as they are susceptible to undermining. The use of cut-off wall at the downstream shoulder, permeable geotextile filter between the embankment fill and the flexible scour protection methods are also possible batter protection options. The Floodway Design Guide [16] further provides design tables for dumped graded rock and gabion mattresses based on range of flow velocities. Special considerations should be given if the velocity exceeds 6.4 m/s when selecting material for dumped graded rock and gabion mattresses.

Floodway protection types

The road drainage manual [15] presents floodway protection methods. Two main categories presented in this floodway design guideline are: grass batters and non-grassed batters. Floodways with grassed batters may be designed if the tail-water level is not more than 300 mm below the downstream edge of the road formation at time of first overtopping. Floodways with other than grassed batters should be used if the above criterion is not satisfied. The road drainage manual [15] informs seven types of floodway protection structures other than the grassed batters. However, details are given for recommended five types only (i.e. type 1, 2, 4, 5 and 7). These five types are also recommended as floodway protection structures in section 4.5 of the Guide to Road Design Part 5B [14]. The floodway design guide [16] presents three types of scour protection designs taken from a MRWA rural road-upgrading project.

3.1.2.6. Other considerations

Table 25 outlines some other factors that need to be considered during the floodway design process.

Table 25: Other Considerations

	Guide to Road Design Part 5: Drainage [14]	Road Drainage Manual [15]	The Floodway Design Guide [16]
Environmental factors	Floodways reduce risk of scour to waterways and surrounding land Floodways should be designed in a manner to reduce the	Outlines basic environmental factors to be considered	Outlines four main areas under Environmental Impact, Construction Effects, Channel Modification, and Ethnographic Issues

	period of submergence due to ponding or backwater		
Expected fish migration during times of flood	Check the allowable flow velocities for fish migration	Check the allowable flow velocities for fish migration	Provision of adequate water flow to downstream areas is required
Fauna and terrestrial passage through culverts	Fauna movement should consider if only floodplain culverts are provided. Designers only need to consider terrestrial movement.	Refers to the details provided in the Guide to Road Design Part 5: Drainage [14]	Provision of adequate water flow to downstream areas is required

3.1.2.7. Recent Floodway Types used in Lockyer Valley area

The longitudinal profile of a floodway recently constructed in the Lockyer Valley Regional Council area is shown in Figure 1. Cross sections vary along the longitudinal direction of the road segment for a given floodway. Different floodway sections are found along the length of the floodway based on the ground profile of floodway sites. Figure 1 shows four zones labelled as A, B, C for with no culverts and D for floodways with culvert. These typical sections are shown in Figure 2- Figure 5.

These typical floodway cross-sections are highly site specific. Depending on the site condition, only one or two cross-sections may be adopted or upstream rock protection may be omitted. Different pavement types are also another possible variation.

Figure 1 - Figure 5 are extracted from the Left Hand Branch Road Drawing Schedule issued for construction on 17th September 2014 [8-10].

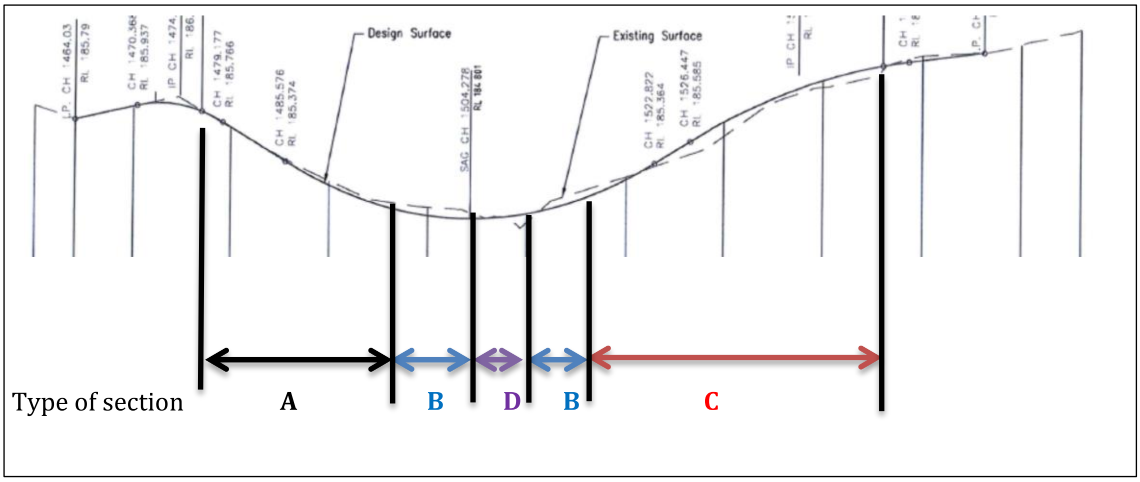


FIGURE 1: FIGURE INDICATING ZONES WITH DIFFERENT FLOODWAY SECTIONS [17]

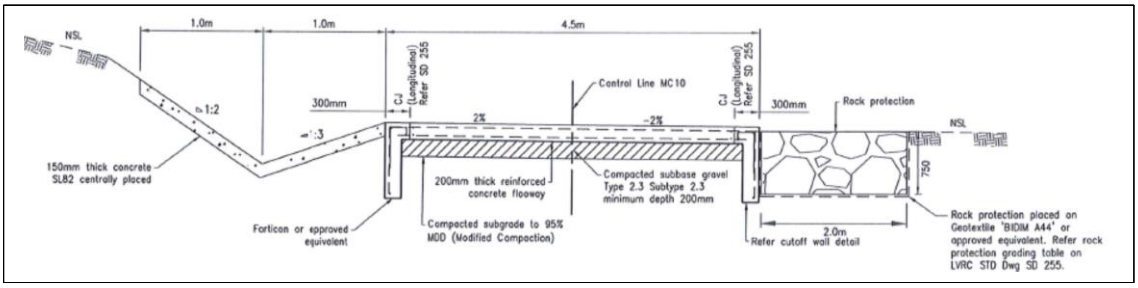


FIGURE 2: TYPICAL CROSS-SECTION FOR SECTION A [18]

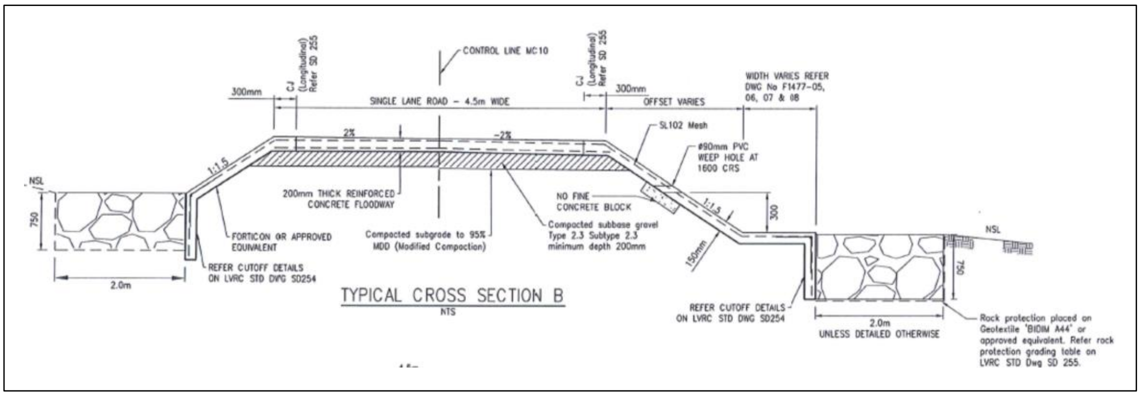


FIGURE 3: TYPICAL CROSS-SECTION FOR SECTION B [18]

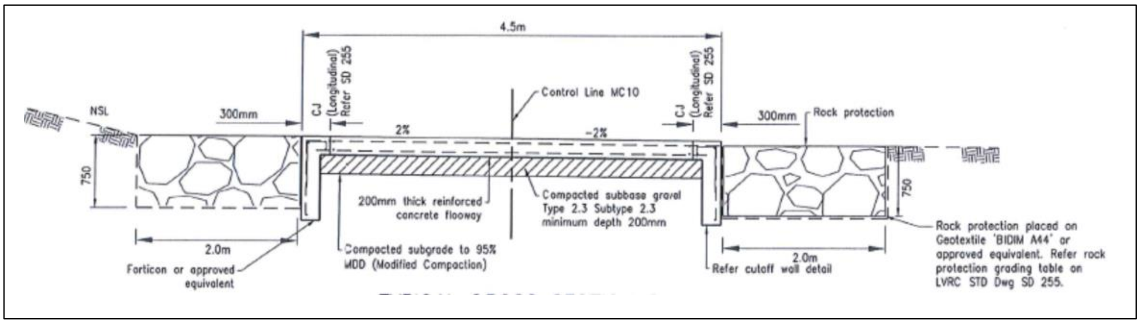


FIGURE 4: TYPICAL CROSS-SECTION FOR SECTION C [18]

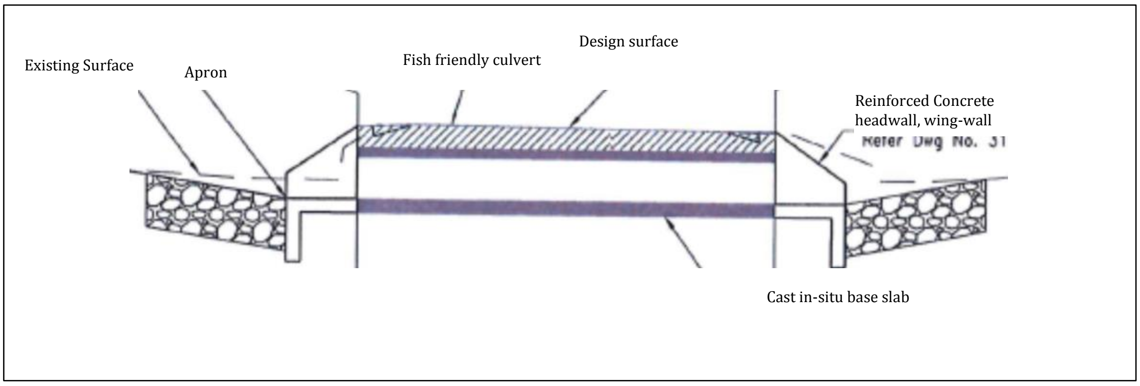


FIGURE 5: TYPICAL CROSS-SECTION FOR SECTION D (WITH CULVERT) [19]

3.1.2.8. Novel floodway types

The final report for floodway research conducted by GHD Pty Ltd [20] reviews possible alternative products commercially available for floodway construction. These alternative products are: concrete canvass, granular polymer based product such as Polycom® and SoilTrac® soil stabiliser, concrete blocks, and cement treated products. Some councils in the South Australia had agreed to trial some of these alternative products in their floodways as listed below.

1. PolyCom®: A floodway in the Gumbowie Road in the District Council of Peterbrorough
2. Concrete Canvas®: A floodway in the Caroona Road in the Regional Council of Goyder
3. Concrete blocks: A floodway in the Yednalue Road in the Flinders Ranges Council
4. Roadway cement stabilization: A floodway in the District Council of Orroroo
5. Rock rip-rap trial: A floodway in the Tarcowie Road in the Nothern Areas Council

3.1.2.9. International Floodway Guidelines

Afghanistan Engineer District (AED) Design Requirements: Culverts & Causeways [21] outlines hydraulic design aspects for any project requiring drainage structures across the roadway, including culverts and floodways or causeways. It recommends the use of causeways for situations where the peak runoff is less than 2.2 cubic meters per second or runoff less than the capacity of a 1m x 1m reinforced concrete box culvert. This guideline outlines criteria that a contractor should satisfy when designing a causeway. This also provides basic geometric properties for a causeway.

For larger causeways, a detailed design process considering the hydraulic aspects of the site is recommended. Two main types of floodways are discussed based on the terrain of the road segment (i.e. flat or mountainous). Flat terrain is defined as the causeways with channel slope less than 3 degrees. If the channel slope is in excess of 3 degrees, causeways will be treated as in a mountainous terrain. Geometrically, all causeways shall be more than 10 m in length and shall be sloped to a low point approximately in the center of the longitudinal alignment. 10% longitudinal slope is recommended along the centerline of the road segment. Heaving stone revetments are recommended to protect embankments, upstream and downstream ends of a causeway. The length of the revetment may vary based on the type of causeway. The embankment gradation should be designed based on the average approach channel velocity. Small diameter PVC or HDPE bypass pipes are recommended for causeways crossing irrigation canals or washes with continuous runoff. These bypass pipes should be provided beneath the compacted backfill of the causeway slab between the upstream and downstream weep holes with a standard spacing of 2 meters. Speed of approaching vehicles should be controlled by speed bumps at approach side with necessary warnings.

In addition to general geometric requirements, AED Design Requirements: Culverts & Causeways [21] presents use of debris control devices in the upstream zone. Debris control devices should be used if the tributary area of the causeway has potential of carrying large amount of debris. If the upstream tributary area of the causeway has high potential for boulders, cobbles and gravel sediment, an energy dissipation type debris structure shall be provided. Such control mechanisms should be capable of reducing the volume of material transported across the road surface. An example of a debris control method is shown inFigure 6.

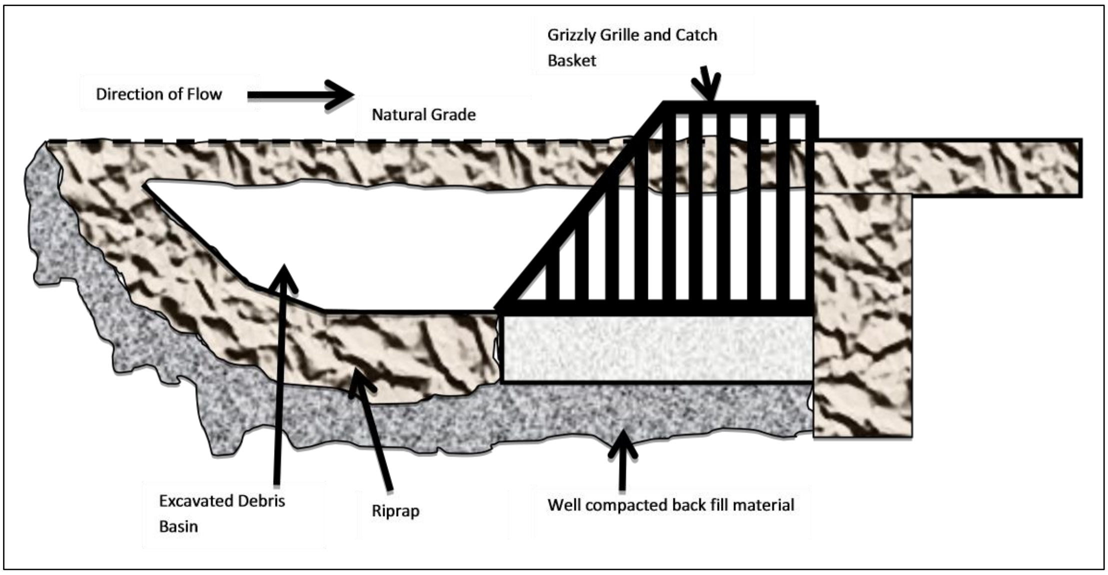


FIGURE 6: USE OF GRIZZLY CAGE AND DEBRIS BASIN TO CONTROL THE EFFECT OF DEBRIS [22]

3.2. DESIGN LOADS & LOAD COMBINATIONS

3.2.1. BRIDGES

3.2.1.1. AS 5100

AS 5100 Bridge Design code (Section 15 of AS 5100.2-2004) [11] gives relevant equations to calculate the flood induced forces on bridges resulting from water flow, debris and log impact.

Forces due to water flow

When the bridge superstructure is partially or fully inundated in a flood, it is subjected to a horizontal drag force (F_d^*) normal to its longitudinal axis and a vertical lift force (F_L^*) as given in AS 5100.

Drag force

$(F_d^*) = 0.5C_d\, V^2 A_s$

where:

C_d is the drag coefficient read from the chart given in the code;

V is the mean velocity of water flow (flood);

A_s is the wetted area of the superstructure, including any railings or parapets, projected on a plane normal to the water flow.

Lift force

$(F_L^*) = 0.5C_L\, V^2\, A_L$

where:

C_L is the lift coefficient read from the chart given in the code;

V is the mean velocity of water flow (flood);

A_L is the Plan deck area of the superstructure.

Moment on superstructure

According to AS 5100 [11], drag and lift forces generate a moment about the longitudinal axis of the superstructure. The resulting moment at the soffit level at the centre-line of the superstructure shall be calculated as follows:

$M_g = 0.5C_m V^2 A_s d_{sp}$

where:

C_m is the moment coefficient and varies from 1.5 to 5 depending on the relative submergence of the superstructure.

Forces due to debris

Debris load acting on superstructures is given by the code as,

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

where:

C_d is the drag coefficient read from the chart given in the code;

V is the mean velocity of water flow (flood);

A_{deb} is the projected area of the debris mat described in the code.

Forces due to moving objects

According to AS 5100 [11], where floating logs or large objects are a possible hazard, the drag forces exerted by such logs directly hitting bridge girder (superstructure) shall be calculated on the assumptions that a log with a minimum mass of 2 tons will be stopped in a distance of 75 millimetres for such solid girder (superstructure). A draft revision of the AS 5100 [23] suggests consideration of the "large item impact" in urban areas, where large floating items such as pontoons, pleasure craft, shipping containers etc. can impact the bridge structure. However, the code suggests that forces due to log impact or large item impact debris shall not be applied concurrently on the structure.

F_{log} shall thus be given by the following equation.

$F_{log} = mV^2/2d$

Where:

m is the mass of the log or the impacting object;

d is the stopping distance specified by the code (eg. 0.075m for solid concrete piers);

V is the velocity of the water (m/s).

3.2.1.2. Eurocodes

Eurocode 1 [24], Part 1.7 considers flood, fire and earthquake as accidental effects and has suggested a risk analysis to be undertaken for such events. Following introduces some forces affecting bridges due to an event of flood.

Forces due to water flow

Eurocode 1, Part 2.6 [25] considers actions due to water during execution into two categories: static pressures and hydrodynamic effects. The magnitude of lateral water force to bridges is given by (Figure 7):

$$F_{wa} = k\rho_{wa}hbv_{wa}^2$$

where:

v_{wa} is the mean speed of the water, averaged over the depth, in m/s;
ρ_{wa} is the density of water in kg/m3 ;
h is the water depth, but not including, where relevant, local scour depth in meters;
b is the width of the object in meters;
k is the shape factor:
k = 0.72 for an object of square or rectangular horizontal cross-section,
k = 0.35 for an object of circular horizontal cross-section.

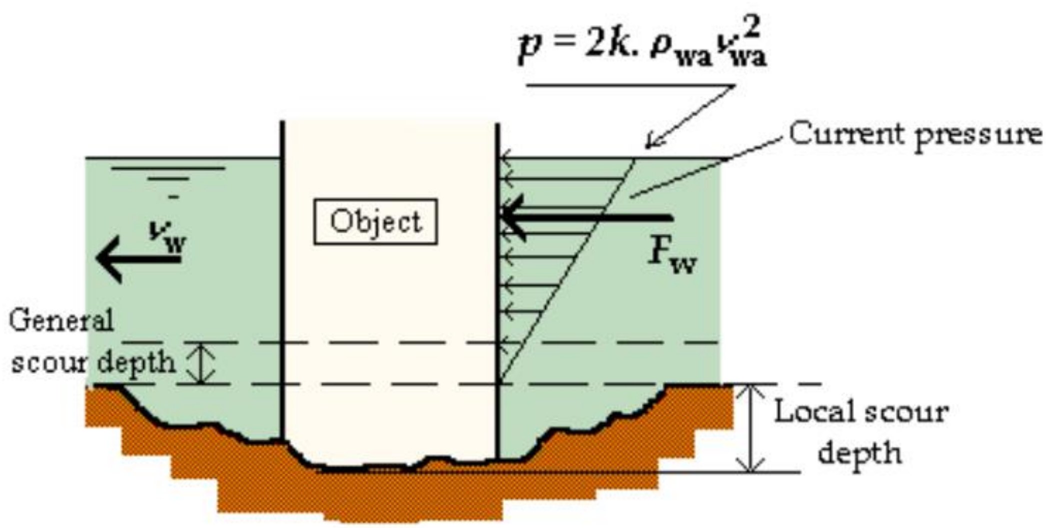
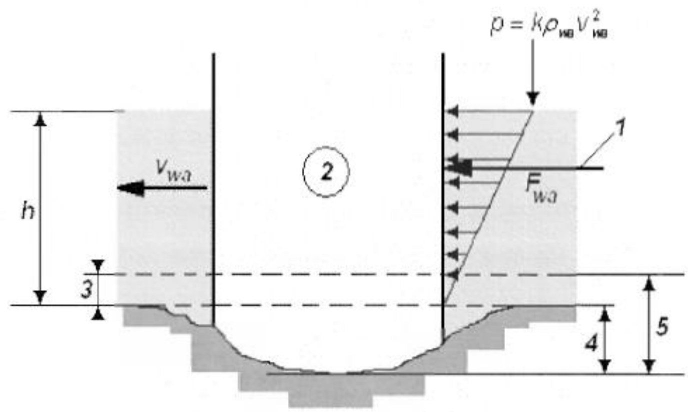


FIGURE 7 PRESSURE AND FORCE DUE TO CURRENTS ON BRIDGE PIERS [25]

Interestingly, Eurocode 1, Part 1.6 [26] introduces the above formula with a minor difference, multiplying 0.5 to the formula, as follows (Figure 8):

$$F_{wa} = \frac{1}{2} k \rho_{wa} h b v_{wa}^2$$



Key

1 Current pressure (p)
2 Object
3 General scour depth
4 Local scour depth
5 Total scour depth

FIGURE 8 PRESSURE AND FORCE DUE TO CURRENTS [26]

However, the values of shape factor (k) have been doubled accordingly, which will result the same water force, as follows:

k = 1.44 for an object of square or rectangular horizontal cross-section,

k = 0.7 for an object of circular horizontal cross-section.

Eurocode 1 [26] also notes that a more refined formulation can be used to determine the water force for individual projects.

Forces due to debris

According to Eurocode 1 [26], debris force F_{deb} should be calculated using the following formula:

$$F_{deb} = k_{deb} A_{deb} v_{wa}^2$$

where:

k_{deb} is a debris density parameter, in kg/m^3 (recommended value is 666 kg/m^3) ;

v_{wa} is the mean speed of the water average over the depth, in m/s;

A_{deb} is the area of obstruction presented by the trapped debris and falsework, in m^2.

3.2.1.1. American Standards

AASHTO LRFD [12] categorises the water loads (W_A) into 4 categories: static pressure, buoyancy, stream pressure and wave load. Similarly, ASCE [13] categorises the water loads into hydrostatic and hydrodynamic loads in where, wave loads are categorised as a special type of hydrodynamic loads. ASCE also mentions the Impact loads result from objects transported by floodwaters striking against structures and their components. The stream pressure has been further categorised into: longitudinal and lateral in AASHTO [12].

Hydrostatic loads

ASCE defines hydrostatic loads the ones caused by water either above or below the ground level, which is either still or moves at velocities less than 1.52 m/s. These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts [13]. These loads are further divided into vertical downward, upward and lateral loads depending on the geometry of the surfaces and the distribution of hydrostatic pressure.

Longitudinal forces

The longitudinal forces on substructures which are similar to the drag forces mentioned in Australian standards are calculated as follows:

$$p = \frac{C_D V^2}{1{,}000}$$

where,

p is the pressure of flowing water (ksf);

C_D is the drag coefficient for piers, which can be read fromTable 26;

V is the design velocity for the design flood in strength and service limit states and for the check flood in the extreme event limit state (ft/s).

Type	C_D
Semicircular-nosed pier	0.7
Square-ended pier	1.4
Debris lodged against the pier	1.4
Wedged-nosed pier with nose angle 90 degrees or less	0.8

TABLE 26 DRAG COEFFICIENT [12]

However, AASHTO [12] also refers to the theoretically correct formulation for calculation of the drag force as follows:

$$p = \frac{C_D w V^2}{2g}$$

where,

w is the specific weight of water (kcf);

C_D is the gravitational acceleration constant 32.2 (ft/s^2);

V is the velocity of water (ft/s).

AASHTO asserts that the floating logs, roots, and other debris which may accumulate at piers and, by blocking parts of the waterway, need to be considered and provides a New Zealand Highway Bridge Design Specification provision as a design guidance.

Lateral forces

AASHTO [12] also introduces the lateral forces which are uniformly distributed pressure on substructures due to water flowing at an angle, θ, to the longitudinal axis of the pier Figure 9.

$$p = \frac{C_L V^2}{1{,}000}$$

where,

p is the lateral pressure (ksf);

C_L is the lateral drag coefficient, which depends on the angle θ as shown in the following figure and table.

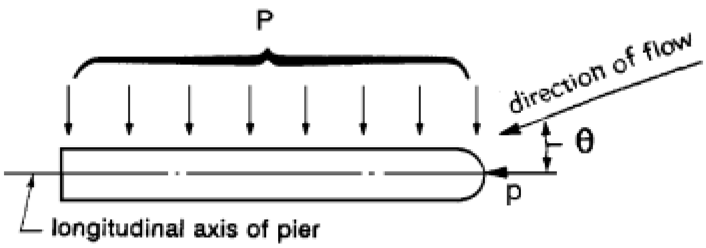


FIGURE 9 PLAN VIEW OF PIER [12]

Angle, θ, between direction of flow and longitudinal axis of the pier	C_L
0 degrees	0.0
5 degrees	0.5
10 degrees	0.7
20 degrees	0.9
≥30 degrees	1.0

TABLE 27 LATERAL DRAG COEFFICIENT [12]

Flood velocity

As estimation of flood velocities includes a variety of epistemic uncertainties, FEMA [27] suggests a lower and upper bound for the estimation of flood velocities in design in coastal areas (Figure 10), which are given as follows:

$V = \frac{d_s}{t}$ Lower bound

$V = (gd_s)^{0.5}$ Upper bound

where,

V is the flood velocity (m/s)

d_s is the Stillwater flood depth (m)

t is 1 second

g is the gravitational constant (9.81 m/s^2)

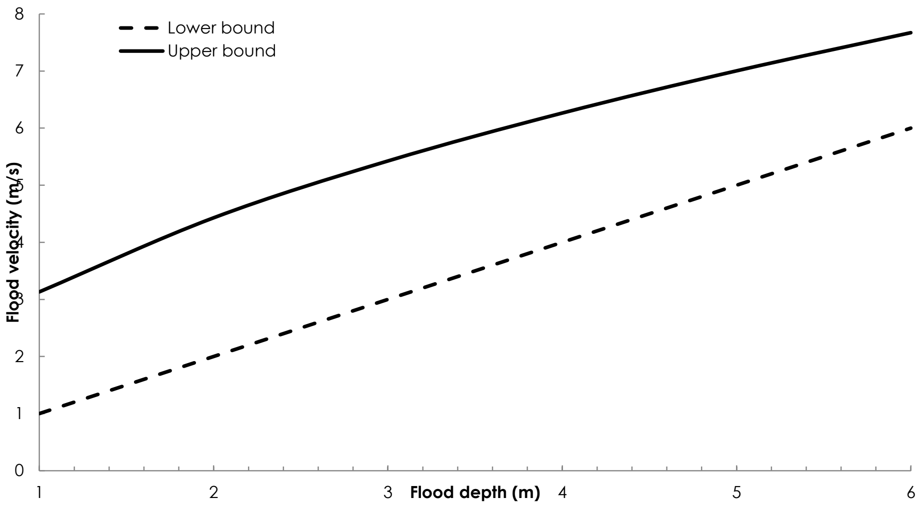


FIGURE 10 DESIGN FLOOD VELOCITY

Impact loads

ASCE [13] categorises the impact loads into 3 categories: normal impact loads, special impact loads and extreme impact loads which are depending on the frequency and the size of the object impacting the structure. ASCE suggests that "*given the short-duration, impulsive loads generated by flood-borne debris, a dynamic analysis of the affected building or structure may be appropriate. However, in some cases (e.g., when the natural period of the building is much* *greater than 0.03 s), design professionals may wish to treat the impact load as a static load applied to the building or structure."[13]*. Therefore, the following formula has been suggested for estimation of the force [13].

$$F = \frac{\pi W V_b C_I C_O C_D C_B R_{max}}{2g\Delta t} \qquad \text{(C5-3)}$$

where

F = impact force, in lb (N)
W = debris weight in lb (N)
V_b = velocity of object (assume equal to velocity of water, V) in ft/s (m/s)
g = acceleration due to gravity, = 32.2 ft/s^2 (9.81 m/s^2)
Δt = impact duration (time to reduce object velocity to zero), in s
C_I = importance coefficient (see Table C5-1)
C_O = orientation coefficient, = 0.8
C_D = depth coefficient (see Table C5-2, Fig. C5-1)
C_B = blockage coefficient (see Table C5-3, Fig. C5-2)
R_{max} = maximum response ratio for impulsive load (see Table C5-4)

3.2.2. FLOOD-WAYS

Tthe floodway design process is traditionally governed by the hydraulic design aspects. Design loads and load combinations are not discussed in floodway design guidelines. However, floodways are subjected to different forces during its service life. These force components include self-weight, traffic loads, soil pressures and forces resulting from water flow. During extreme flood events, floodways are closed for traffic and hence traffic loads can be neglected when assessing flood damage. However, forces resulting from water flow need to be considered. Those forces depend on the flood characteristics such as flood discharge, velocity and depth as well as the density of water. Applicable forces due to water flow are identified as drag, lift, debris and log-impact for floodways. In the absence of specific details of these fluid forces for floodways, the AS5100.2 was followed (Refer section 3.2.1) by considering the worst case scenario.

Several assumptions are required to derive drag and lift coefficients, mainly due to the difference of orientation and flow profiles around floodways. Therefore, an ANSYS Fluent study is being conducted concurrently for a detailed investigation. Drag and lift effects are automatically coming from the ANSYS analysis. The preliminary ANSYS results indicate that drag and lift forces are small for floodway structures without culverts. This will be further investigated in the future analysis.

Debris load should be included considering the debris mat AS5100 suggests. However the dimensions given for bridges may not be applicable for floodways. Impact load that is considered in this research is what is suggested in AS5100 for solid concrete pier. This may need modifications. From the visual observations of the damaged floodways in LVRC, it is necessary to consider impact load for floodways.

The way it is calculated for log may not be relevant for floodways and a proper way to account for damage due to moving boulder like objects needs to be investigated.

3.3. ANALYSIS METHODS

3.3.1. BRIDGES

Australian standard [28] states that "analysis for all limit states shall be based on linear elastic assumptions except where nonlinear methods are specifically implied elsewhere in the standard or approved by the relevant authority".

AASHTO [12]accepts any method of analysis which can satisfy the requirements of equilibrium and compatibility and utilizes stress-strain relationships for the proposed materials.

3.3.2. FLOOD-WAYS

Feasibility of using a floodway should be first established considering the facts such as traffic volume, community expectations, frequency of flooding and economical aspects. Floodways are generally used in rural road networks with low traffic volumes. However, community acceptance for partial closures of floodway during flood events should be accounted in conjunction with the frequency of flooding. If the use of a floodway is warranted as the feasible option, the design process should be started. Design of a floodway is mainly based on hydraulic aspects. However, considerations should be given to geometric and safety requirements and community needs. Therefore, floodway design process includes hydrological analysis, hydraulic design, geometric and safety considerations and community expectations.

3.3.2.1. Hydrological investigation

Hydrological investigation is an essential part in the floodway design process to obtain design flow values and hydrographs at different flood intensities. The floodway design guide [16] outlines three methods for the hydrological analysis. The rational method and the index method are recommended for small catchments whereas the runoff-routing modelling is recommended for larger catchments in excess of 50 km^2. The RORB software program is commonly used in the Western Australia for the runoff-routing modelling. However, the designer should adjust design flows to match with observed or historical data. Therefore, the designer should check the availability of gauged flood data. The hydrological analysis further involves developing hydrographs to plot change in discharge with time for a given flood event. Runoff-routing analysis and/or measured data should be used for large catchments. Anecdotal information and/or observed data are commonly used for small catchments. These hydrographs will be useful to estimate the time of closure and submergence. The road drainage manual [15] states that minimum of 20 years of recorded data is required when using stream gauge data to achieve acceptable level of accuracy.

The hydrological investigation and hydraulic design process requires reliable field survey data to calculate the floodway capacity, velocities, upstream and downstream water levels. The floodway design guide [16] outlines minimum required field survey data for a floodway as: a cross section(s) across the river, a long section along the streambed and a long section on the road centerline. A detailed contour plot is also recommended to help the designer, but is not essential.

The floodway design guide [16] highlights that backwater and upstream flooding should be estimated to identify the affected upstream distance, especially in case of existence of upstream property or infrastructure. Floodway should be designed to minimize the backwater effects when the upstream assets cannot cope with the increased flood levels. In Western Australia, Department of Water (DoW) and Water Investigation and Assessment branch should be consulted with regard to allowable backwater levels.

3.3.2.2. Hydraulic design

Australian floodway design guidelines outline a simplified and a detailed method for hydraulic design.

Summary of main steps of the detailed hydraulic design method are outlined below. The appendix C of the floodway design guide [16] provides flow charts to further elaborate the detailed design process. Figure 11 shows the nomenclature used in floodway analysis.

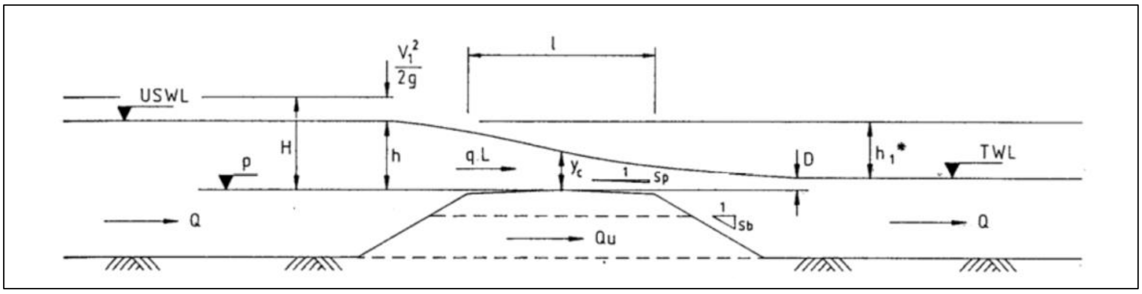


FIGURE 11: FLOODWAY NOMENCLATURE [16]

Step 1: Determine the stage-discharge curve (i.e. elevation vs discharge curve) for the natural section using the Manning's formula given in Equation 1 and basic formula for the discharge Q=AV

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad \text{Equation 1}$$

Where: *V* = flow velocity in m/s

n = Manning's roughness coefficient

A = Cross-sectional area of flow in m^2

R = Hydraulic radius in meters

S = Stream hydraulic gradient in meters per meter

Q = flow rate in m^3/s.

Step 2: Select initial (first trial) floodway crest level and length of floodway (L) as shown in Figure 12 and assume a height of headwater (h) above the floodway crest.

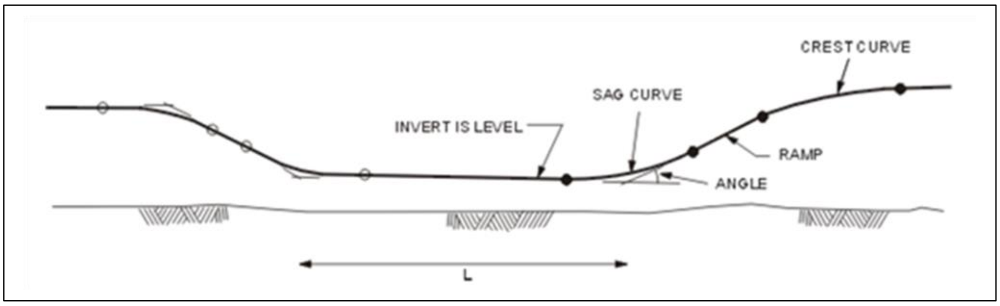


FIGURE 12: LONGITUDINAL SECTION OF A TYPICAL FLOODWAY [16]

Step 3: Calculate H/l

Where: *H* = total head = h + $V^2/2g$

h = height (m) of headwater above floodway crest

V = Average velocity (m/s) of flow approaching floodway

l = floodway width (m)

Step 4: Calculate discharge in m^3/s over floodway using the broad crested weir formula given in Equation 2.

$$Q = C_f L H^{1.5} \left(\frac{C_s}{C_f}\right) \qquad \text{Equation 2}$$

Where:

Q = discharge over floodway in m^3/s

C_f = coefficient of discharge at free flow condition

C_s = Coefficient of discharge for flow with submergence

C_s/ C_f = Submergence factor

L = length of floodway in meters

H = Specific head or specific energy in meters

The coefficient of discharge, C_f and the submergence factor, C_s/C_f are obtained from charts given in floodway design guidelines.

Step 5: (if submergence is present) Check whether the discharge over the floodway matches with the design discharge and adjust the depth of flow above floodway crest (h), the floodway crest level or length and repeat the procedure.

3.3.2.3. Use of culverts

Provision of culverts in a floodway plays a significant role to the floodways analysis and design process. Culverts reduce the discharge over the road segment and also help to increase the tailwater depth at the time of overtopping. Floodways should be associated with drainage culverts, if they are designed as the primary waterways structure. However, drainage culverts can be omitted if the floodway is constructed at ground level. Main functions of drainage culverts are:

- To reduce the backwater
- To raise the tail-water level in order to reduce the head across the floodway and to reduce the batter protection requirement
- To facilitate drainage and prevent ponding behind the embankment as a measure of preventing piping or sediment transportation that can affect the structural integrity
- To facilitate drainage and prevent overtopping for smaller, more frequent flows (i.e. act as an anti-ponding structure).
- The floodway design guide [16] recommends nominal drainage culverts for low-lying floodways, which will be neglected during the hydraulic calculation. Larger culverts and their effect to the discharge should be included in the hydraulic analysis process, especially for the cases with higher embankments.

3.3.2.4. Types of flow over a floodway

Floodway analysis and design process should also pay attention to the possible types of flow over the floodway. Understanding of types of flow over a floodway helps to select appropriate batter protection. Two main types are free flow and submerged flow. Free flow is further sub-divided into plunging flow and surface flow. Plunging flow causes submerged hydraulic jump on the downstream zone and associated with high velocities leading to potential of erosion. Surface flow separates from the surface of the road embankment and rides over the surface of the tail-water. Downstream erosion potential is therefore less. During submerged flow, discharge is controlled by both headwater and tail-water levels.

Table 28 below outlines corresponding sections in three Australian floodway design guidelines for hydraulic design aspects.

TABLE 28: REFERENCES FOR SECTIONS OUTLINING HYDRAULIC DESIGN ASPECTS

	Guide to Road Design Part 5: Drainage [14]	Road Drainage Manual [15]	The Floodway Design Guide [16]
Use of Culverts/ waterway openings	use of waterway openings	use of waterway openings	use of drainage culverts except for the floodways constructed at ground level
Flow regimes Free flow: Plunging flow	Free flows occur at the initial stage. Plunging flow: flows over the shoulder and down the downstream face of the embankment. Produces a submerged hydraulic jump with higher velocities that will be erosive.		

Surface flow	Surface flow: flow separates from the surface of the road embankment and rides over the surface of the tailwater. Erosive potential is less.
Submerged flow	Submerged flow occurs when the discharge is controlled by the tailwater level as well as the headwater levels. Submerged flow occurs when the flow depth over the road is everywhere greater than the critical depth.
Hydraulic Design Methods	All three floodway design guidelines outline a simplified and detail design methods.

3.4.DESIGN PROCEDURE

3.4.1. Bridges

Australian standard [28] covers a 100 years design life for bridges. Therefore, the bridge structure and its elements shall satisfy all limit states during the design life. Limit states are categorised in two categories: 1. Ultimate limit state and 2. Serviceability limit state.

The **ultimate limit states** shall satisfy the following:

$\varphi R_u \geq S^*$

where

R_u is the nominal capacity of the element;

φ is the capacity reduction factor;

S^* is the design action for ultimate and serviceability limit states.

According to Australian standard [28] the ultimate limit states include the following:

"*(a) Stability limit state, which is the loss of static equilibrium by sliding, overturning or uplift of a part, or the whole of the structure.*

(b) Strength limit state, which is an elastic, inelastic or buckling state in which the collapse condition is reached at one or more sections of the structure. Plastic or buckling redistribution of actions and resistance shall only be considered if data on the associated deformation characteristics of the structure from theory and tests is available.

(c) Failure or deformation of any foundation material causing excessive movement in the structure or failure of significant parts of the structure.

(d) Deterioration of strength occurring as a result of corrosion or fatigue, or both, such that the collapse strength of the damaged section is reached. Consideration shall be given to the implications of damage or any other local failure in relation to the available load paths.

(e) Brittle fracture failure of one or more sections of the structure of sufficient magnitude such that the structure is unfit for use."

Australian standard [28] defines the **serviceability limit states** to include the following:

"(a) Deformation of foundation material or a major load-carrying element of sufficient magnitude that the structure has limitation on its use, or is of public concern.

(b) Permanent damage due to corrosion, cracking or fatigue, which significantly reduces the structural strength or useful service life of the structure.

(c) Vibration leading to structural damage or justifiable public concern.

(d) Flooding of the road or railway network, surrounding land and scour damage to the channel bed, banks and embankments."

3.4.2. Flood-ways

Existing floodway design process is based on hydraulic design aspects. The section 3.3.2 and 3.5.2 present the floodway analysis and design procedure and a design example respectively. However, these examples are based on a levelled crossing using a uniform section. Recent floodway types used in the Lockyer Valley Regional Council area (i.e as shown in section 3.1.2.7) are constructed on vertical curves with combination of different sections along the longitudinal direction. This adds complexity to hydraulic design process as the flow over the floodway may significantly vary from estimations made using the Broad-crested weir formula. Three Australian floodway design guidelines do not present a design example for this type of floodways.

Further literature indicates that the flow velocity varies across the depth and the cross sections. Velocity at the deepest section is greater than the mean velocity estimated from the Manning's formula. This can create increased stresses on mid sections of floodways than the sections at the edges. Higher velocities around the deepest section can be estimated by dividing the floodway cross section into small sections and then analyzing. This may add some complexity to the design process, but may increase the accuracy of the prediction.

3.5.SUMMARY

Out of all the 4 available design guidelines for floodways, Afghanistan Engineer District (AED) Design Requirements: Culverts & Causeways [21] gives minimum details on the design process. The Guide to Road Design Part 5: Drainage[14] serves as the national framework to design floodways in Australia. The design procedure outlined in this Austroad publication is similar to that in the Road Drainage Manual [15] published by the Queensland Transport and Main Roads. However, Floodway Design Guide [16] published by the Main Roads Western Australia is very comprehensive in giving the design procedure based on the hydraulic aspect. Loads and load combinations are not outlined in these four floodway design guidelines and hence structural analysis is missing. The design is mainly governed by the hydraulic analysis. However, extreme flood events have caused significant damage to floodways as evident from recent floods in Queensland. Inspection reports indicate that common floodway failure mechanisms are associated with fluid-structure interaction. The following method is proposed to fill the gaps identified in the design guidelines as well as to find solutions to the problems identified by the end users:

1. It is proposed to compare the behaviour of a floodway subjected to flood loadings using two methods: using the bridge design loads from AS5100 and utilising finite element analysis incorporating flood loadings on the structure using ANSYS software. Since it is uncertain whether bridge loads are directly applicable for floodways, forces, stresses and displacements acting on a floodway will be compared with the results obtained from the finite element analysis. It is expected to derive loading functions for drag and lift forces for floodways based on the outcomes of this analysis.
2. Currently Lockyer Valley Regional Council is trialing with the use of cutoff walls in floodways. They have identified previously that it is the most vulnerable area of a floodway. The method in step 1 will be extended to investigate the reinforcement configuration and the sizes of the cutoff walls to make the floodways more resilient.
3. The study will then be continued to identify possible damage areas at different flood intensities. A step wise analysis will be carried out as outlined in Figure 13 & 14. Firstly, damage initiation process will be established based on excessive stresses or displacements or combinations of both as outlined in the Figure 13. This will establish the critical flood intensity that has a potential to damage one or more areas in a given floodway. Then structural response of the floodway will be investigated using a step-wise damage propagation method by introducing damage zones to floodways as outlined in the Figure 14.

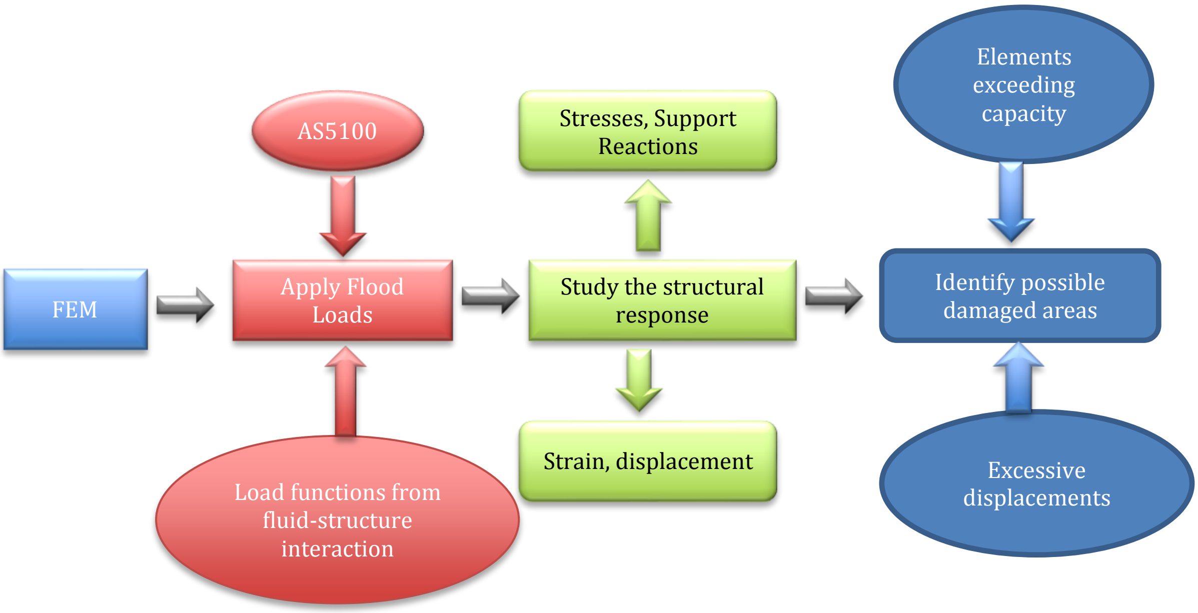


FIGURE 13: IDENTIFICATION OF DAMAGE INITIATION

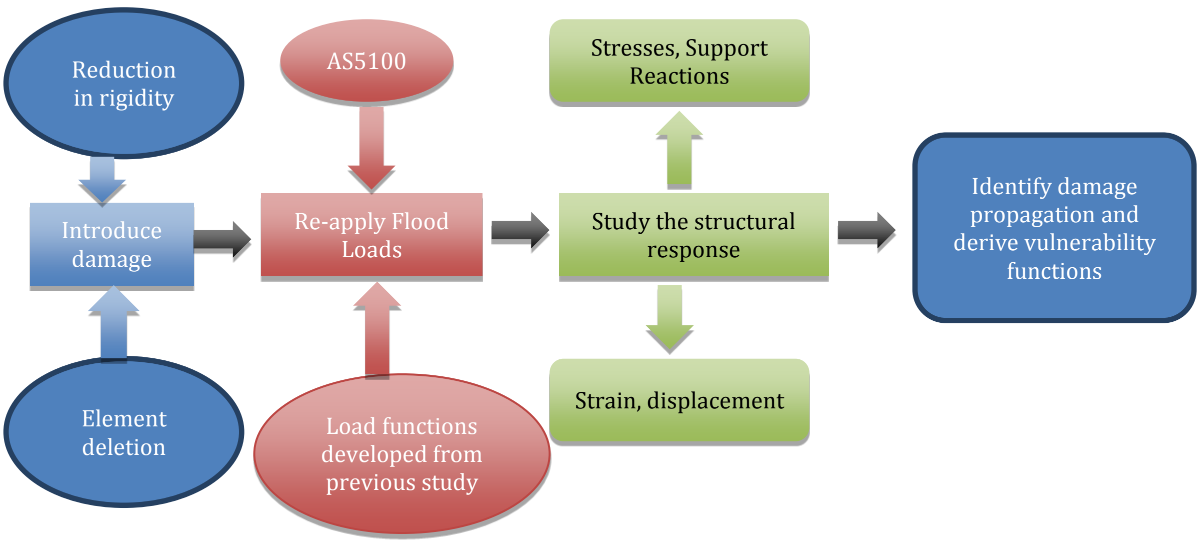


FIGURE 14: IDENTIFICATION OF DAMAGE PROPAGATION

3.6. CASE STUDY

3.6.1. Bridge Design Example

Please refer to the Report no. 3 for this case study.

3.6.2. Floodway Design Example

This section provides step by step guidance on designing a new floodway with a culvert based on the Guide to Road Design Part 5B: Drainage – Open Channels, Culverts and Floodway [14].

3.6.2.1. Design Parameters

The floodway should be trafficable at 20 year annual recurrence interval flood with a discharge of 40 m^3/s at an approaching flow velocity of 0.5 m/s. The floodway is not designed to be super elevated to avoid depth variation in the lateral direction and to minimize the effect of surface debris on the floodway surface. The floodway length (L) is taken as 50 m with surface width (l) of 9 m.

3.6.2.2. Design assumptions

The stage-discharge curve is assumed in this example as further explained below.

The stage-discharge curves at the road centerline and the downstream end should be obtained across uninterrupted sections (i.e. assuming that there are no structures present at the crossing) using the Manning's equation (i.e. Equation 1). This step requires the cross section profiles as well as site inspection reports or photographs for estimating the Manning's coefficient based on the surface condition. The hydraulic gradient 'S' should preferably be estimated from the water surface profile. Alternatively, estimation can be made using the slope of the streambed at the proposed floodway crossing over a relatively long section to avoid the influence of local scouring and sediment transportation effects.

Only one stage-discharge curve may be used if there are no significant changes in hydraulic gradient and the cross section profiles of the natural section between the road centerline and the downstream end. This example is based on a single stage-discharge curve given in Figure 15.

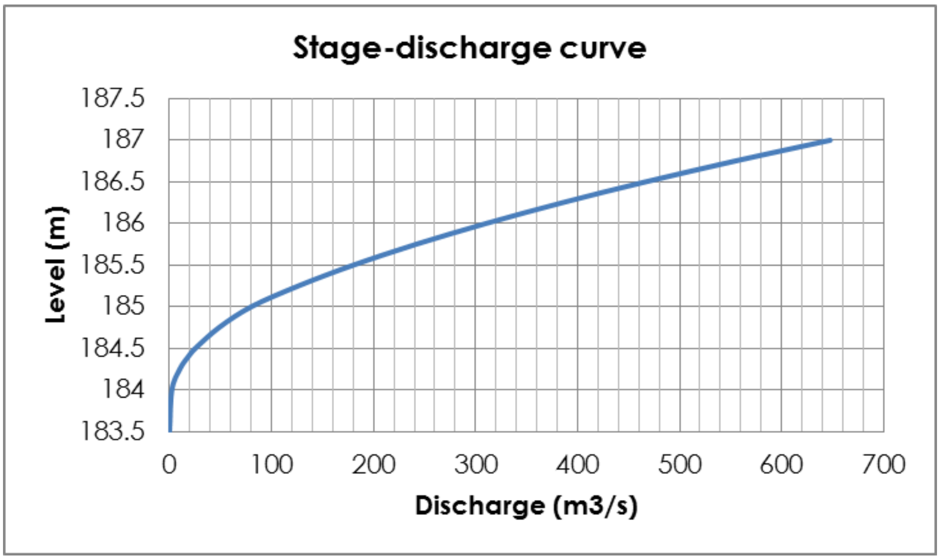


FIGURE 15: STAGE-DISCHARGE CURVE FOR THE FLOODWAY DESIGN EXAMPLE

3.6.2.3. Load/load combinations

Current floodway design process is based on hydraulic design aspects only. Structural loads and load combinations are not outlined in existing floodway design guidelines.

3.6.2.4. Analysis method

Floodway analysis process is carried out based on the hydraulic analysis and design procedures as outlined in the section 3.5.2.5.

3.6.2.5. Design procedure

Step 1: Adopt a road level and calculate the maximum allowable depth of water over the road

The initial road level is taken at the 20 year ARI flood level of the unrestricted section. From the stage-discharge curve, the initial road level is taken as 184.65 m.

As 20 year annual recurrence flood should be trafficable, total head at the upstream end should be less than 0.3 m.

Based on the total head $H = h + V^2/2g \leq 0.3$ m and approaching velocity of 0.5 m/s (refer Figure 12 for notations):

$h = 0.3 – 0.5^2/(2x9.81) = 0.287$ m

Step 2: Calculate the discharge over the road

Using the broad-crested weir formula (given in
Equation 2), the discharge over the road is estimated.

$H/l = 0.287/10 = 0.0287 < 0.15$

The coefficient of discharge, C_f, is obtained from the Chart A of the Figure 4.3 in the Guide to Road Design Part 5B: Drainage – Open Channels, Culverts and Floodway [14].

$C_f = 1.675$

As the road level is assumed to be at the tailwater depth of 20 year annual recurrence interval flood, submergence is neglected (i.e. Cs/Cf = 1).

$$Q = 1.675 \times 50 \times 0.3^{1.5}(1) = 13.76\, m^3/$$

Therefore culverts should be designed to carry the excess discharge of Q_c.

$$Q_c = 30 - 13.76 = 16.24\, m^3/s$$

This floodway at the designed road level at 185.65 m requires culverts to carry 16.24 m^3/s discharge at an operating head of 0.3 m as outlet control condition.

Step 3: Design of the culvert

The culvert design process is based on the procedure outlined in the section 3.10 of the Austroads guide [14].

Selecting a trial culvert size

Let's assume that Reinforced Concrete Box Culverts (RCBC) will be used.

Height of the culvert opening, D, is given by:

D = Crown level of road – crossfall – minimum fill above culvert – thickness of deck slab – invert level

D = 184.65 – 0.02 x 4.5/2 – 0.1 – 0.2 – 183.5

D = 0.805 m

Assumed values are: 2% two way crossfall, 100 mm fill above the culvert and 200mm thick deck slab.

Assume the maximum velocity, V_{max}, through the culvert as 2.5 m/s.

Required culvert waterway area, A:

$$A = Q_c/V_{max} = 16.24/2.5 = 6.496\ m^2$$

Design discharge for trials

Assuming 8/1200 x 750 culverts and 10m in length

Height of culvert, D = 0.75m

Nominal box width, B = 1.2m

Discharge per cell, Q = Q_c/8 = 16.24/8 = 2.03 m^3/s

Inlet control headwater depth:

Using nomograph given in Figure B1, HW/D = 1.7

HW_i = 1.7 x 0.75m = 1.275m

Outlet control headwater depth:

Assuming entrance loss coefficient, k_e, 0.2,

Using the nomograph given in the Figure B5:

H = 0.37m

Critical depth, d_c:

$$d_c = 0.467\ (Q/B)^{\left(\frac{2}{3}\right)} = 0.467(2.03/1.2)^{\left(\frac{2}{3}\right)} = 0.66\ m$$

For 20 year ARI flood, tail water depth, TW = 1.15 m > D

Therefore, h_0 = TW = 1.15 m

Assuming culvert slope, S_0 = 1% = 0.01

Headwater level under outlet control condition, HW_0,

$$HW_0 = H + h_0 - L\,S = 0.37 + 1.15 - 10\ \times 0.01 = 1.42\ m$$

Allowable headwater level = 1.45 m

Determining the controlling headwater:

HW_i = 1.275 m

HW_o = 1.420 m

$HW_i < HW_o$

HW_o Controls

Outlet velocity – outlet control

$V_o = Q/A = 2.03/0.9 = 2.26$ m/s

Outlet Froude Number

Culvert is full as this is outlet control situation

$$F_r = V_o/\sqrt{gy} = 2.26/\sqrt{9.81 \times 0.75} = 0.833 < 1$$

This is subcritical flow

There is no hydraulic jump at this stage

Design Check

V_o (=2.26 m/s) is less than V_{max} (=2.5 m/s).

Therefore the design is satisfactory for ARI 20 year flow.

However, culvert should be checked when the flood is at the point of overtopping (i.e. upstream water level equals to the maximum road level). Also afflux for ARI 50 year flow should be calculated and checked.

3.6.2.6. Discussion of the outcome

The floodway is capable to support ARI 20 year flow with use of 8/1200x750 RCBC at a road level of 184.65 m. This is the instance of first overtopping. At this situation, the tailwater level is above the edge of the road formation and hence grass batters can also be used. However, the decision on selecting the batters should be made in lieu with other conditions, such as time of submergence, frequency and intensity of flood events and scour and erosion potential in the site. Further, time of submergence and closure should be checked for the designed maximum flood event by accounting the community needs.

3.7. DISCUSSION AND RECOMMENDATIONS

Report No 3 on Community resilience to road network disruption, emphasizes the importance of more resilient floodways in the Lockyer Valley region. Normally the designer is allowed to make his/her own decision based on design calculation in conjunction with economical aspect and expected service level for the serving community. At the moment, design of floodways is mainly based on the hydraulic aspect with little consideration given towards structural aspects and community impact. Therefore it is important to formulate a design process for floodways which takes into account structural as well as hydraulic aspects together with the Importance of the community resilience.

Currently, Lockyer Valley Regional Council is trialing with different cut-off wall arrangements to improve the performance of floodways. Therefore, a comprehensive study based on detailed finite element analysis incorporating potential cut-off wall options is needed in order to give recommendations for the design of floodways.

4. BUSHFIRE EFFECTS ON STRUCTURE PERFORMANCE

4.1.DESIGN STANDARDS ANALYSIS

4.1.1. BRIDGES

Fire is one of the most severe environmental hazards to which build infrastructures are subjected to .Fire on infrastructure could cause significant loss towards the personal, financial and quality of life in a country. Codes and standards supply methods, experiences and possible measures for achieving better fire safety. Standards referenced in codes, in broad term can be classified in to 3 categories namely material standards, engineering practice standards and testing standards. Standard fire codes can be of prescriptive based codes or performance based codes. A performance based fire resistance approach determines the evolution of the structural capacity once it undergoes a realistic fire condition. Compared to other codes used in North America and Australia Eurocodes are much more progressive in adapting the performance based fire resistance analysis of structures. Design provisions given in Eurocodes are well received by the engineering and scientific community. They are referenced by other codes worldwide. Following table summarized the list of codes available. So in the discussions in the following chapters are more towards the recommendations given in Eurocodes.

TABLE01: CLASSIFICATION OF THE EUROCODES [29]

Eurocode number	*Ambient conditions*	*Fire conditions*
Basis of design	EN 1990	–
Actions	EN 1991-1-1	EN 1991-1-2
Concrete structures	EN 1992-1-1	EN 1992-1-2
Steel structures	EN 1993-1-1	EN 1993-1-2
Composite steel-concrete structures	EN 1994-1-1	EN 1994-1-2
Timber structures	EN 1995-1-1	EN 1995-1-2
Masonry structures	EN 1996-1-1	EN 1996-1-2
Geotechnical design	EN 1997	–
Earthquake resistance	EN 1998	–
Aluminium alloy structures	EN 1999-1-1	EN 1999-1-2

In recent years, due to the rapid development of urban ground transportation systems, as well as increasing transport of hazardous materials (such as flammable materials, spontaneously combustible and poisonous materials) bridge fires have become a concern. In addition, the bridges in bush fire prone areas have a significant risk of getting exposed to different levels of fire conditions. Bridge fires can lead to significant economic and public losses.

For structural members, fire safety objectives are achieved through fire resistance provisions. Fire resistance is the duration during which a structural member exhibits acceptable performance with respect to structural integrity, stability and temperature transmission.

In a review work on the literature of bridge fires in America[30] it states that the American NFPA 502: Standard for Road Tunnels, Bridges, and Other Limited Access Highways states that, ‘‘Protection of Structure – Critical structural members shall be protected from collision and high-temperature exposure that can result in dangerous weakening or complete collapse of the bridge or elevated highway’’.

But there is no guidance is given on how to protect bridges from fires that can result in ''dangerous weakening or complete collapse''. Similarly, the European standard Eurocode 1 part1.2 deals with the traffic loads to be considered in bridges[31] and does not contain any provision related to how fire hazard should be taken into account in bridge design. Even European fire related codes omit bridge fires as they describe ''the thermal and mechanical actions for the structural design of buildings''.[32]

The intensity of a bridge fire depends on type, quantity of fuel and ventilation characteristics. Since bridges are generally located in open zones, there is no dearth of oxygen for fuelling the fire. The fire behaviour of bridge girders can be significantly different from that of beams in buildings due to different loading, geometry, and sectional characteristics. Therefore, the available fire-resistance information from building structures cannot directly be applied to bridge girders[32]This signifies the importance of developing suitable fire curves for bridge fire analysis. Different types of fire curves developed for building applications can be found inEurocode 1 part 1.2[31]; they will be discussed in preceding sections.

Accuracy of a numerical analysis of the structural system heavily depended on the use of proper material constitutive equations (not fairly established) and temperature dependent material constant (fairly established). A review and a comparison of the literature related to the stress-strain constitutive equations of concrete material at elevated temperature could be found in [33]. A more complete account of constitutive modeling can also be found in Eurocode 2 part 1.2[34] and Eurocode 3 part 1.2[35] for steel and concrete.

4.2.DESIGN LOADS & LOAD COMBINATIONS

4.2.1. BRIDGES

In the event of a fire the most likely applied loads are much lower than the maximum design loads specified for normal temperature conditions. Most of the codes do not consider wind snow or earthquake loading at the same time as the fire.

TABLE 02 DEAD AND LIVE LOAD FACTORS FOR FIRE DESIGN

	Dead load	Permanent load	Other live load
Eurocode	1.0Gk	0.9Qk	0.5Qk
USA(ASCE)	1.2Gk	0.5Qk	0.5Qk

In US The ASCE-07(2005)[36] is used to calculate the loading on the building. According to the British standards Eurocode 1 action on structures can be used to the calculate the loadings on structures. To calculate the fire and the heat flux induced in the elements by the fire in particular Eurocode 1 part 1.2[31] should be used.

Thermal load on to a structure is specified in terms of a fire temperature curve. Standard time temperature curves keep increasing with time whereas the actual fire temperature curve is decreases with time after reaching a certain maximum, which is more realistic. Bridges are more likely to expose to hydrocarbon fires. So it is suitable to use hydrocarbon based standard or actual time temperature curve for bridge analysis. However there are no explicit hydrocarbon fire curves designs for bridge application. On the other hand time temperature curves to simulate the bush fire scenario should be developed in the future.Figure presents three different nominal fire curves given inEurocode 1 part 1.2. Temperature here represents the gas temperature near the steel member. It is advised in the code the external curve is not meant to be used for the design of external steel structures.

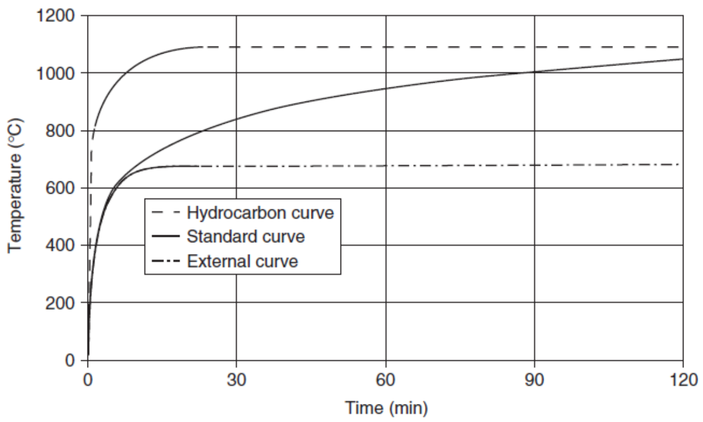


FIGURE 01 THREE DIFFERENT NOMINAL FIRE CURVES AS SPECIFIED IN EUROCODE[29]

Many other methods given in the code is not suitable for the application of bridge fire evaluation. Eurocode 1 part 1.2Annex D allows using the computational fluid dynamic models to compute the thermal action on structures as long as the computational method obeys the conservation laws of physics.

Another most popularly used standard time temperature curves are given in ISO 384 and ASTM E119. These curves are used heavily by US[29].

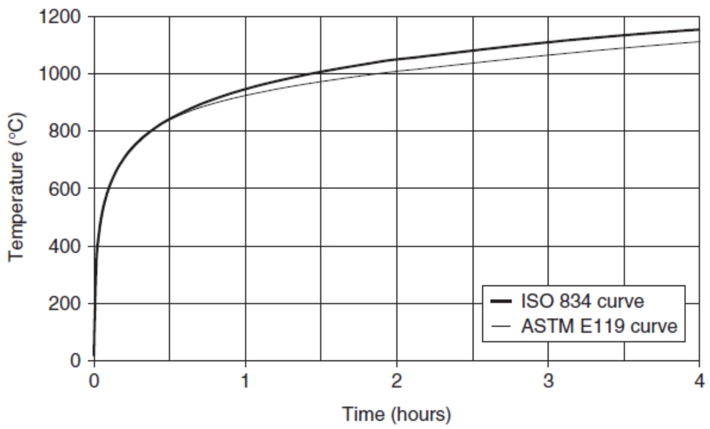


Figure 02 Comparison between ISO 834 and ASTM E119 fire curves[29]

Heat exchange coefficients are given in the code to calculate the net heat flux reaching a steel member accounting different exposure conditions.

Current version of Australian standard AS5100.2 -2004 Bridge design(design loads)[37] for bridge loading does not contain a section that covers bushfire and hydrocarbon based fire loading on bridges.

4.3. ANALYSIS METHODS

4.3.1. BRIDGES

Structural analysis of bridges subjected elevated temperature is complicated by the effects of elevated temperatures on the internal forces and the properties of material. According to the Eurocode 2 part 1.2[34] analysis of the structure can be done locally or globally. In the case of local analysis, the part of the structure to be analysed , the relevant failure mode in fire exposure, the thermal expansions and deformations such , that their interaction with other parts of the structure can be approximated by time independent support and boundary conditions during fire exposure. When a global structural analysis for the fire situation is carried out the relevant fire exposure, the temperature dependent material properties and member stiffness's, effects of thermal expansions and deformations shall be taken in to account.

Hand calculations can only be performed with simple structural forms with simple support conditions and uniform internal temperature. Taking a finite element approach is one of the popular ways in calculating the load carrying capacity of a structural system subjected to elevated temperature. There are number of proprietary computer programs for this purpose. Most popular programs that can be found in the literature are SAFIR, VULCAN, VecTor3. In addition generic FE programs that can be used for this purpose includes NASTRAN, ANSYS, ABAQUS. Those computer programs can handle the material and geometric nonlinearity of the structure once undergoes an extreme loading event.

Analysis of a structure exposed to fire required to calculate the deformation of the structure under applied load. Deformation of structure is directly related to the change in strain. Normally 4 different strain components can be found to amount the total strain. The accuracy of an analysis depends on calculating these 4 different components.

$$\varepsilon_{c_{tot}}(\sigma, T, t) = \varepsilon_{c_\sigma}(\sigma, T) + \varepsilon_{c_{th}}(T) + \varepsilon_{c_{cr}}(\sigma, T, t) + \varepsilon_{c_{tr}}(\sigma, T)$$

Four different components in the above equation respectively are stress related strain (instantaneous), free thermal strain, creep strain and transient strain. Transient strain component only presents in concrete. It is related to the phase transformation of the concrete during the first time heating and will not be present in the subsequent heating and cooling cycles.

4.4. DESIGN PROCEDURE

(This section does not need to contain details of all formula and factors used in the code. Only outline and highlights)

4.4.1. Steel

4.4.1.1. Bridges

There are 3 main steps in a fire resistance analysis of steel structures.

1. Determine the fire temperature-time relationship particular to a given fire scenario.
2. Calculate the temperature history in the steel structure due to the fire temperature
3. Undertaking the structural analysis considering the effect of step 2 considering additional static or dynamic loading.

Step one of the procedure is discussed previously in section 4.2.1. The most important input in the second step of the procedure is use of high temperature thermal properties of steel and insulation materials. ASCE manual 1992[38] and Eurocode 3 part 1.2[35]provides that information in detail. AISC(American Institue of steel construction) manual is the principle reference in US for steel steel structure design. AISC manuel provides some information on thermal and mechanical properties at elevated temperatures[29]. Thermal elongation, conductivity and specific heat are the properties that are interested. Though the trends are the similar there are slight differences in value between different codes in different temperature ranges. Thermal properties of fire protection materials are limited in literature; good account could be found in SFPE handbook of Kodur and Harmathy[39].

TABLE 03 THERMAL PROPERTIES OF COMMONLY USED FIRE PROTECTION MATERIAL[29]

Material	*unit mass* ρ_p [kg/m^3]	*mosture content p* [%]	*thermal conductivity* λ_p [w/(m · k)]	*specific heat* c_p [J/(kg · K)]
Sprays				
– mineral fibre	300	1	0.12	1200
– vermiculite cement	350	15	0.12	1200
– perlite	350	15	0.12	1200
High-density sprays				
– vermiculite (or perlite) and cement	550	15	0.12	1100
– vermiculite (or perlite) and gypsum	650	15	0.12	1100
Boards				
– vermiculite (or perlite) and cement	800	15	0.20	1200
– fibre-silicate or fibre-calcium-silicate	600	3	0.15	1200
– fibre-cement	800	5	0.15	1200
– gypsum board	800	20	0.20	1700
Compressed fibre boards				
– fibre silicate, mineral-wool, stone-wool	150	2	0.20	1200
Concrete	2300	4	1.60	1000
Light weight concrete	1600	5	0.80	840
Concrete bricks	2200	8	1.00	1200
Bricks with holes	1000	–	0.40	1200
Solid bricks	2000	–	1.20	1200

Eurocode 3 part 1.2[35]deals with the load bearing capacity of elements in steel structures in the event of a fire. It allows calculating duration a structure that could withstand a given loading situation during a fire. Code uses ultimate limit state design philosophy. There is no explicitly mentioned deformation (or deflection limit) criteria that is related to the span of the element. According to the provisions given in Annex E deformation controlling could be archived by using 0.2percent of proof strength instead of the effective yield strength.

Structural analysis can be done in different levels such as global structural analysis, member analysis and sub structure analysis. In the case of member and substructure analysis, no precise recommendation is given in the Eurocode 3 part 1.2to define the boundary conditions at the separation of those entities from the global structure. Eurocode specifies 3 different calculation models to determine the fire resistance of structure and its elements. First mode of calculation used tabulated data. In this method fire resistance time is expressed as a function of set of parameters such as thickness of insulation in a steel section, load level, dimension of a section. Tabulated data are based on the empirical observation and experimental results but not on the basic principles and equilibrium equations. Second model of calculation is based the equilibrium equations called the simple calculation models is aimed for used in everyday calculations without using sophisticated numerical software. This type of calculation is the direct extrapolation of the calculation of a steel member at room temperature to reflect the temperature effect on the yield strength and the Young's modulus of steel at an elevated temperature situation. This type of procedure is applicable only because the material models in the Eurocode 3 part 1.2the creep is considered to be implicitly incorporated in to the stress strain curves. Third category of calculation model is called the advanced calculation model that should be based on the recognised principles of structural mechanics. These types of calculation models can be applied with any type of time-temperature curve provided that appropriate material properties are known. Analysis takes care of the indirect actions of fire in to account. So it is suitable to use in global structural analysis.

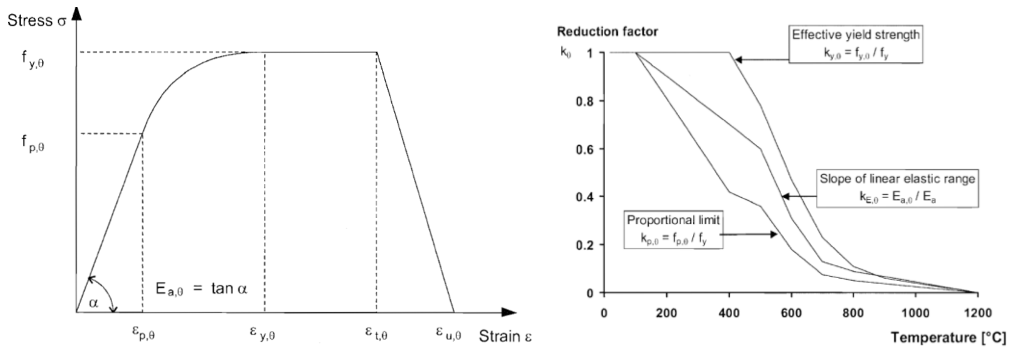


FIGURE 03 STRESS STRAIN RELATION SHIP FOR STEEL AND STRENGTH REDUCTIN FACTORS [35]

ASCE code contains simplified equations based on the standard fire resistance tests for calculating the fire resistance of steel structural members. These empirical methods based on the assumption that rate of temperature rise in a member depends on its weight and the surface area exposed to heat. Unit weight to section perimeter ratio (W/D) of various steel sections and configurations some values are given in AISC manual (2005)[29]. In some cases of evaluating the fire resistance of steel a critical temperature is defined. A critical temperature is the temperature at which the steel loses 50% of its yield strength to that of room temperature. Commonly used limits for the critical temperature for columns and for beams are 538 °C and 593 °C respectively.

Stability analysis of a steel structure can be evaluated thorough three different approaches according toEurocode 3 part 1.2. It could be evaluated in the time domain, in the load domain or it is possible in temperature domain. Following figure summarizes it in detail for a standard fire case.

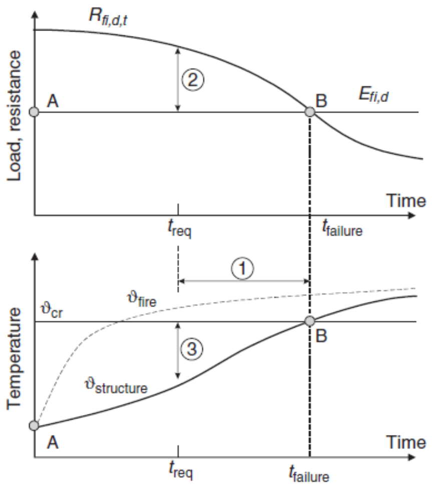


FIGURE 04 LOAD,TIME OR TEMPERATURE DOMAIN FOR A NORMAL FIRE[29]

In the time domain for the safety of the structure it needs to be verified that the time to failure is higher than the fire resistant time. In the load domain at the required time the load resistance of the structure should be larger than a critical load resistant value specified. In the temperature domain at the required time of fire resistant the temperature of the member should be lower than the critical temperature.

Connections in steel structures play an important role by facilitating the transferring of forces or moments of one member to another. Fire resistance of the structure is highly dependent on the extent of redistribution of forces from highly stress region to a less stressed region through connections, which are of the types bolted or welded. However many codes have no explicit fire resistance requirement for connections. Instead they should be protected to the same degree of fire resistance to that of the connecting members. However Eurocode 3 part 1.2[35]sets rules for designing of both welded and bolted connections at elevated temperature thorough the introduction of strength reduction factors. Eurocode 3 part 1.2Annex D provides a way to compute the temperature gradient of a structural joint depending on the depth of members. This temperature distribution can be used to find the strength reduction factors for the strength of bolts and welds by referring to the following table. For further detail refer to the works of Kirby (1995)[40] and Latham and Kirby (1990)[41]

TABLE 04 STRENGTH REDUCTION FACTORS FOR BOLTS AND WELDS[35]

θ_a [°C]	$k_{b,\theta}$	$k_{w,\theta}$
20	1,000	1,000
100	0,968	1,000
150	0,952	1,000
200	0,935	1,000
300	0,903	1,000
400	0,775	0,876
500	0,550	0,627
600	0,220	0,378
700	0,100	0,130
800	0,067	0,074
900	0,033	0,018
1000	0,000	0,000

AS5100.6-2004 Bridge design steel and composite construction code does not explicitly included provisions on fire resistance design of steel and composite bridges. Instead it recommends AS4100 code where it is considered to necessary for a bridge to be designed for fire resistance for ex. Bridges near railway stations.

4.4.2. Concrete

4.4.2.1. Bridges

Provisions for evaluating the fire resistance of concrete members are generally specified in codes and standards. These provisions are derived based on the standard fire tests. Fire resistance is often related to member dimensions or other influencing factors. In this chapter, three most widely used codes are compared briefly, namely, ACI 216.1[42](1997) Eurocode 2 part 1.2(2004)[34] and AS 3600(2009)[43].Though none of these codes directly dealt with bridge design it is recognized that provisions given in these codes shall be applied to such purposes. However AS 5100.5-2004 Bridge design (Concrete)[44] states that buildings are typically designed for non-hydrocarbon fires it may not be applied to many fires that may occur in road and railroad networks. On the other hand AS 3600-2009 code claims that the fire resistance criteria given in section 5 in the code have been revised to take in to account the latest developments inEurocode 2 part 1.2[34]. That reveals the interdependency of different codes.

ACI 216.1[42] specifies a minimum width and a cover to non prestressed and flexural reinforcements in beams to achieve up to 4hrs of fire resistance. This requirement differs according to the restrained and unrestrained support conditions. However a proper definition for the restrained and unrestrained conditions is not given. Does the restrained mean an axially, rotationally or both is not clear. Specifications given in the code is only applicable for normal strength concrete. There is no clear guidance given for the fire resistance evaluation of high strength concrete. Minimum cover to the reinforcement has been specified for slabs according to the restrained and unrestrained conditions. Cover depths also depend upon the aggregate type namely siliceous, carbonate, and semi lightweight and light weight. This standard cannot be used to evaluate fire resistance of the slabs with metal deck floors. AS 3600-2009[43] describes two methods for the evaluation of fire resistant periods (FRPs). In the first methodology designers can refer to the tabulated data and figures in the code. In such case no additional checks is needed for on shear and torsion capacity. Alternatively FRPs can be calculated by the methods of calculation where reference should be made to Eurocode 2 part 1.2.[34] In Australian code, AS 3600-2009 the fire resistance periods for structural adequacy is presented separately for simply supported and continues beams in two charts. Minimum width and the cover should be selected as a pair for a given period of fire resistance. Maximum fire rating is 4hrs according to the code. Australian code does not specify a guide line for the case of axially restrained beams. Minimum thickness for a solid slab designed for a 4hrs fire resistance should not be less than 175mm. In contrast to both ACI 216.1 and AS 3600 where design for fire has been based on prescriptive methodology Eurocode 2 part 1.2allows for performance based design methodology. Eurocode 2 part 1.2 describes three methods of design: a tabulated method, simplified calculation method and a general calculation method. Eurocode 2 part 1.2 provides material properties that can be used in analysis of structures subjected to elevated temperatures. Those characteristic values can be used for advanced and simplified methods given in the code. Code suggests strength reduction factors of concrete and steel at elevated temperature as shown below.

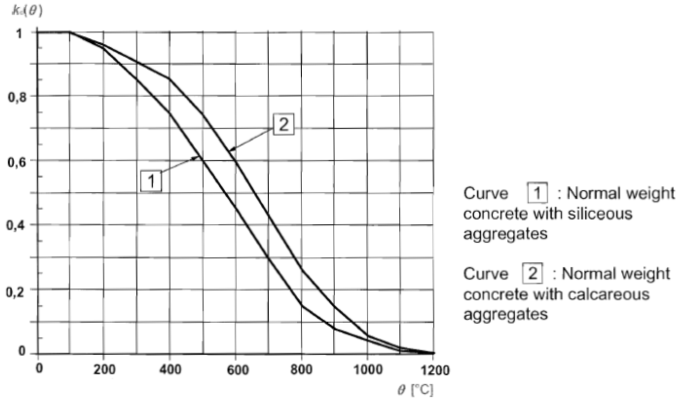


FIGURE 05 CHARACTERISTIC STRENGTH REDUCTION FACTORS OF CONCRETE[34]

Code also allows using alternative material laws as long as they are within the range of experimental evidences. Since creep behaviour of material is not explicitly considered Eurocode material models should be used for the cases where heating rate is 2-50K/min.

Eurocode 2 part 1.2 section 4.3.3 presents platform for an advanced calculation of fire resistance of concrete should be based on in contrast to a tabulated or simple calculation methods. It should acknowledge the principles and assumptions of the theory of structural mechanics taking in to account the changes of mechanical properties of with temperature. Effect of thermal and mechanical induced strains should be duly accounted (thermal strain, stress dependent strain, creep strain and transient state strain). Where relevant the geometric non linearity should be taken in to account.

Explosive spalling of concrete subjected to elevated temperature is another phenomenon that is observed in concrete structure which is related to the moisture content. According to Eurocode 2 part 1.2 explosive spalling should be avoided or its influence on the performance should be taken in to account.

Transient creep strain is a very important factor to be included in to the constitutive relationships for concrete at high temperature. However the whether it should be taking explicitly in to the formulation is not clearly defined. In the Eurocode 2 part 1.2 uniaxial concrete material model, the transient creep is included implicitly in the formulation. In implicit models, the total strain is considered to be comprised of mechanical strain and free thermal strain basically. Transient component of the strain is embedded in the mechanical component of strain implicitly. So the in such models the sequence of heating and loading does not give a difference on the response. On the other hand transient strain could be recovered during an eventual unloading which is not true. So it seems it is important to consider the transient strain explicitly in to account.

AS 3600-2009[43] code allowed to consider an increased fire resistant period provided that the structural members are insulated with acceptable forms of insulations. Minimum thickness of the insulating material to attain the required fire resistance level should be tested according to AS 1530.4[45]. Sprayed or trowelled insulating materials exceeding 10mm should be reinforced to prevent the detachment during a fire.

The upcoming revision on the AS5100[44] will explicitly include a bridge design methodology for hydrocarbon fire. Section will include the information about design material properties at elevated temperatures(100C to1200C)

CHARACTERISTIC COMPRESSIVE STRENGTH REDUCTION FACTORS

Temperature (°C)	0	100	200	900	1200
Reduction factor	1.0	1.0	0.95	0.09	0

CHARACTERISTIC YIELD STRENGTH OF TENSILE REINFORCEMENT

Temperature (°C)	0	100	500	700	1200
Reduction factor	1.0	1.0	0.6	0.1	0

According to the proposed Australian code at the temperature 1200C compressive strength of concrete and yield strength of reinforcements completely lose their strength. In addition Coefficient of thermal expansion, thermal conductivity of concrete minimum tensile strength of tendons, modulus of elasticity will also be provided.

4.4.3. Timber

According to the AASHTO guidelines[46], the fire retardant treatments shall not be applied unless it is demonstrated that they are compatible with the preservative treatment used. Use of fire restarted treatments is not sometime recommended because the large sizes of timber components typically used in bridges have inherent fire resistance characteristics. The pressure impregnation of wood products with fire retardant chemicals is known to cause certain resistance and stiffness losses in wood.

4.5.CASE STUDY

4.5.1. Structure of the proposed cased study

In this case study a performance based study will be conducted covering following areas

1. What are the realistic bridge fire models?
2. Conducting heat transfer analysis
3. What are the available constitutive material models in the literature which accounts the temperature dependent behavior at elevated temperature for concrete, steel, steel-to concrete interfaces?
4. How can we model the composite action of a steel girder bridge?
5. Calculate the significance of high-temperature creep of steel and concrete.
6. Investigating the ways to improve the fire resistance of bridges by introducing passive fire protection measures.
7. Calculating the vulnerability index for a given configuration of a steel bridge?

4.6. DISCUSSION AND RECOMMENDATIONS

Euro code is widely used for the structural fire performance calculations. Eurocodes are refered by many other local and international codes for the calculations of fire performance. Compared to other codes used in North America and Australia Eurocodes are much more progressive in adapting the performance based fire resistance analysis of structures. Many fire international and local codes dealing with calculations of structural fire performance is still rely on prescriptive based approach, where recommendations are based on the experience of the past and or the experimental results. However the validity of such methods applied to complex structural forms where explicit fire performances are needed from the safety point of view is questionable and may not be applicable. In such cases designers need to go for performance based evaluation methods as is prescribed by codes. Results obtained from performance based approach are much more realistic. According to the literature there are no time-temperature curves developed for the application of bridges. Loading and geometrical, evironmental aspects of bridges are completely different from buildings. As a result fire curves developed for the application of building may not be applied for the performance evaluation of bridges unless it is reasonably justified. So there is a compelling need of developing fire curves explicitly to account the hydrocarbon and bushfire situations of bridges.

5. STRENGTHENING OF RC MEMBERS

5.1. DESIGN STANDARDS ANALYSIS

(ACI 440.2R-08, fib Bulletin 14, HB 305-2008 and DR AS 5100.8:2016)

The design calculation procedures of each standard are based on the guidelines prescribed in the corresponding design standards for normal RC elements applied in the particular country or region. The following sections provide only summary and general discussions between the recommendations and guidelines provided by the different standards. The reader is advised to refer to the respective design guidelines for normal structural members and cited research articles for the detailed discussions and procedures. In addition, reference was made to certain sections and clauses of the strengthening design standards throughout the discussion in this document. However, the reader is still advised to refer to the full design standards for the detailed design procedures and equations. It should be mentioned that the discussion presented from DR AS 5100.8:2016 is based only on the draft version. The implementation of the guidelines mentioned and discussed herein can be used only after the final approval of the standard. Amendments on some design guidelines or recommendations may be made in the final approved version of the standard.

Due to the widespread worldwide increase in the use of FRP materials in recent decades, the standards mainly focused on the use of FRP materials for strengthening applications. The design standards referred to the published test results to support the different guidelines and recommendations. The European and American standards (*fib* Bulletin 14 and ACI 440.2R-08 [47, 48]) provided the design procedures using only FRP materials while the Australian standard (HB 305-2008 [49]) discussed the use of FRP and steel plates. The draft version of DR AS 5100.8:2016 [50] provided a discussion on the different strengthening measures that can be implemented for bridges. This included the assessment and repair of bridge bearings, deck joints and bridge barriers.

In addition, the ACI 440.2R-08 and *fib* Bulletin 14 discussed the strengthening design and applications for laminates and/or plates bonded to beams using only adhesives whereas the HB 305-2008 addressed the mechanical bonding of the plates (i.e. bolts). Table 1 compares between the different design topics covered by the design standards. As can be seen from the table, both ACI 440.2R-08 and *fib* Bulletin 14 discussed the strengthening design guidelines for most structural conditions except the mechanical shear connectors. Prestressed FRP and torsional load design was only addressed by the *fib* Bulletin 14 while the HB 305-2008 provided detailed design considerations for the mechanical shear connectors using bolts. Only the DR AS 5100.8:2016 discussed the condition assessment and repair of the bridges superstructures bearings and joints.

In general, the basic ultimate limit state requirements for the design of FRP strengthened elements are based on the assumption that the FRP material is an additional reinforcement and the structure should be capable of carrying a reasonable amount of load in the accidental loss of FRP reinforcement.

This is achieved in the ACI 440.2R-08 by reducing the safety factors of the imposed and dead load in the ultimate limit state design (1.1 for the dead load and 0.75 for the imposed load) as explained in section 9.2 of the standard and described by:

$$(\phi R_n)_{existing} \geq (1.1 S_{DL} + 0.75 S_{LL})_{new} \qquad \text{Equation 1}$$

This accidental situation is considered by the *fib* Bulletin 14 using safety factors of 1.0 for the materials and the reduced load combinations and coefficients depending on the type of accidents (section 3.1.2.5). Reference should be made to the Eurocode 1 Part 1 [51] for obtaining the necessary coefficients and safety factors.

The strengthening design standards recommend the use of different reduction factors for FRP materials at the different environmental conditions. These factors are necessary for the calculation of the ultimate strength of the materials used for design. Table 2 summarizes the safety factors given by the ACI 440.2R-08, DR AS 5100.8:2016 and *fib* Bulletin 14. The ACI 440.2R-08 and DR AS 5100.8:2016 provide the different values of reduction factors based on the exposure condition of the FRP strengthened member. As can be seen from the table, the reduction values are relatively lower at the exterior exposure conditions. The *fib* Bulletin 14 provides the reduction factors based on the type of application of the FRP materials. The table shows that the FRPs that are applied using the wet lay-up methods are more prone to environmental deterioration. This can be attributed to the variations of properties that result from the application of the wet lay-up systems on the site. In addition, Table 2 shows that the reduction factors for glass fibres are the lowest due to the lower durability properties of this type of fibres. The HB 305-2008 recommended the use of the environmental reduction factors given by the ACI 440.2R-08.

The structures that are carrying a constant load under certain adverse environmental conditions are susceptible to creep-rupture. In general, carbon and aramid fibres have a better creep-rupture resistance than the glass. The ACI 440.2R-08 specifies limits for the stress in FRP under cyclic and fatigue loads in order to prevent the failure due to the creep or fatigue rupture (Clause 10.2.9 and Table 10.1). The stress in the FRP should not exceed 0.2, 0.3 and 0.55 of the ultimate tensile stress of the glass, aramid and carbon fibres, respectively. However, the *fib* Bulletin 14 suggests that these values are 0.8, 0.3 and 0.5 for the carbon, glass and aramid, respectively (Section 4.6.2). Although both standards referred to the same references [52, 53], the ACI 440.2R-08 reduced the values with an imposed factor of 1/0.6. The stresses in the concrete and steel should also be controlled under service loads. The limiting factors of the steel and concrete are specified by both standards to be 0.80 and 0.45, respectively. However, the *fib* Bulletin 14 recommends the use of factor 0.60 for the rare load combination such as seismic loadings. According to ACI 440.2R-08, the strain level in the steel reinforcement should be limited to 0.005 for non-prestressed strengthened members as specified by the ACI 318 [54] to provide adequate ductility (Clause 10.2.7 in the ACI 440.2R-08). This value was specified by the *fib* Bulletin 14 to be limited to 0.0043 for class of concrete below C35/45 and 0.0065 for classes higher than C35/45 (Section 3.3). In addition, the compression zone depth should be limited in order to provide a sufficient curvature at ultimate. This is expressed in the *fib* Bulletin 14 with reference to the EC2 as the ratio of compression zone to the effective depth. This ratio should be limited to 0.45 and 0.35 for concrete classes lower and higher than C35/45, respectively. This is also expressed in terms of the FRP strain in which the strain should be more than 0.005 and 0.0075 for concrete classes up to and lower than C35/45 and C35/45, respectively. The recommendations given in section 9.2 of HB 305-2008 state that the use of thin steel plates, NSM or side plates can provide larger curvature prior to debonding of the plate or bar which enhances the ductility of the strengthened beams. The standard also explained two approaches to evaluate the ductility of the strengthened beams. The first approach is only applied in the beams where the concrete crushing occurs before the debonding of plate. This approach depends on the assumption that the beam can develop plastic hinges near the supports and hence allow for the discontinuity of the slope (Clause C9.2.1). The second approach is applied in the case where the debonding occurs at early stage, i.e. before concrete crushing. The beam in this case is considered to be still in the elastic stage. Hence, the ductility of beams can be evaluated by calculating the difference in the flexural rigidities magnitudes along the beams length using either plane frame analysis or equivalent flexural rigidities methods (Clause C9.2.2).

The control failure modes defined by the ACI 440.2R-08 are concrete crushing in the compression zone (before the yielding of tension steel), rupture of FRP after the yielding of steel, concrete crushing after the yielding of steel, separation of concrete cover at the level of steel reinforcement and FRP debonding (Clause 10.1.1). Figure 1 illustrates the different debonding failure modes defined by the ACI 440.2R-08. The concrete crushing failure mode occurs when the amount of compressive strain in concrete is 0.003 while the FRP ruptures at its ultimate tensile strain. However, the debonding failure occurs when the substrate of concrete cannot sustain the stresses in the FRP. In addition, the intermediate crack-induced debonding can be restricted by limiting the strain level in the FRP. This limit is provided as the value of FRP failure strain times 0.9 (Equation 10-2). The ACI 440.2R-08 also recommends the use of the transverse clamps to prevent such a failure. This can be achieved by the use of U-wrap FRP. The value of ε_{fd} can be between $0.6\varepsilon_{fu}$ to 0.9_{fd} in the case of NSM FRP which depends on some factors such as, the dimensions of the member, ratios of steel and FRP and the FRP bar surface roughness. However, the standard recommends the value of $0.7\varepsilon_{fd}$. Different limitation criteria for the different modes of failure are given by the *fib* Bulletin 14. The standard recommends different verification approaches that prevent the undesirable failure modes such as, the end debonding and critical diagonal crack (Section 4.4). These approaches are based on the determination of the sufficient bond length such that it can provide a better anchorage and limitation of the stresses in the FRP. The stress limit is determined based on the tensile stresses at each crack. The minimum length of the bonded FRP or steel plate necessary to prevent the intermediate crack debonding is given in equation 4.1.2(2) in HB 305-2008. The standard also provides recommendations for extending the length of the plate beyond the required minimum length. This approach used for calculating the extended anchorage length is based on the moment redistribution of the member (Clause 4.2.1.2). In addition, the standard recommends the use of different bonding techniques to prevent the shear intermediate crack debonding failure (Clause 4.2.3). These techniques include full wrapping, U-wrapping or jacketing and side bonded plates as illustrated in Figure 4.2.3 in the standard. In addition, section C5 in the standard provides detailed design guidelines for the bolted plates. This includes the prevention of different failure modes in the bolted plates such as longitudinal and post-splitting (Sections 5.3 and 5.4). The buckling resistance calculation of FRP and steel plates is also discussed in section 7 in the standard. The DR AS 5100.8:2016 provides overall guidelines and procedures for assessing the condition of the bridge bearings, deck joints, barriers and culverts (Sections 7 to 9). The repair options including rehabilitation, replacing, concrete pad repair, and pressure wash of barriers are presented. The second section in Appendix A (section A2) provides a brief review of the properties of the different FRP types, namely, carbon laminate, carbon fabric, glass fabric and aramid fabric. The requirements for the adhesives used for bonding the FRP materials are discussed in section A.2.3. This includes the primer, adhesives for CFRP laminates, saturating resins and putty filler. The procedures and guidelines for installing the FRP are detailed in section A3. The standard draft shows similar environmental safety factors to those recommended by the ACI 440.2R-08 and presented in Table 2 in this report. In addition, the standard provides the basic principles of the capacity of FRP strengthened beams as either determined based on recommendations given in AS 5100.7, or by using equation A6.2.3 (For compressive strain < 0.003). Similar failure modes to those presented in ACI 440.2R-08 are adopted by the DR AS 5100.8:2016. All ductility requirements and reduction safety factors should be in accordance with AS 5100.5 and AS 5100.7. In addition, equation A6.3.5.2 provides a strain limit for the intermediate crack (IC) debonding. This equation is the same as that provided by ACI 440.2R-08 (Equation 10-2). The limit for the shear stress in the longitudinal direction is given by equation A6.3.5.3(1) in the DR AS 5100.8:2016. Consideration should be given to the cracks and yielding of steel reinforcement at FRP tips and large shear forces sections. However, an equation for the shear stress is specified in equation A6.3.5.3(2) for the cases where the FRP laminates are not tapered. Clause A6.3.6.3 specifies a value of $0.7\varepsilon_{fu}$ as the value of strain at IC debonding for the near surface mounted (NSM) strengthened beams. However, the cover separation limits are similar to those given for the externally bonded FRP and mentioned above. The serviceability requirement for creep and fatigue for different types of FRP are provided in Table A6.4.1 in the standard. The CFRP, GFRP and AFRP have stress limits of 0.55, 0.20 and 0.20 of their ultimate strength, respectively.

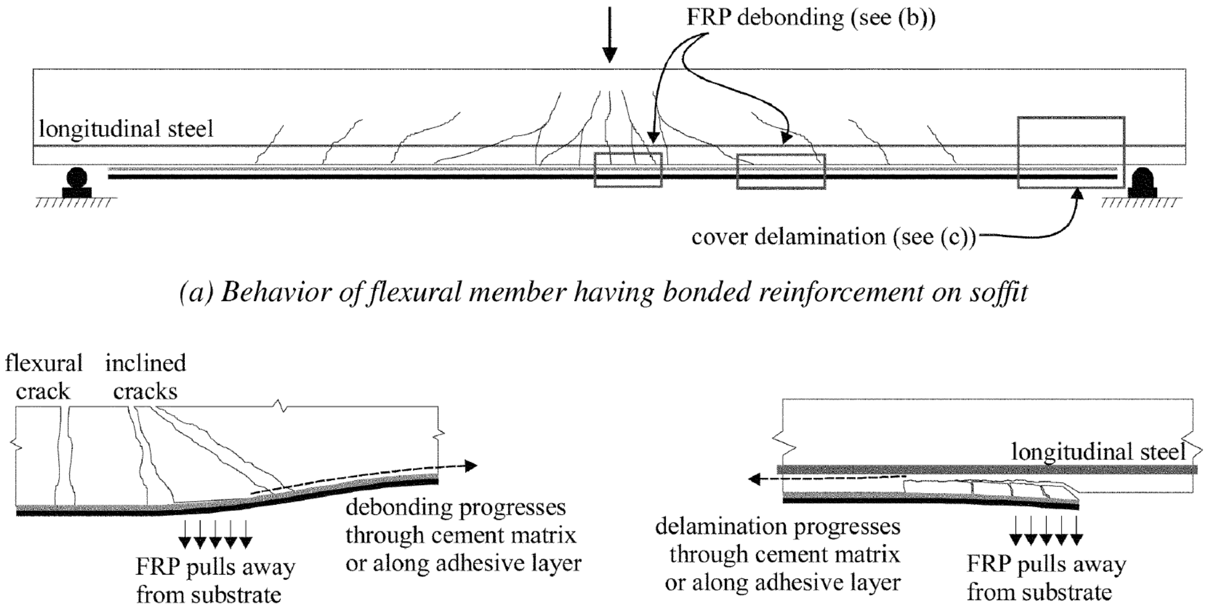


FIGURE 1: DEBONDING FAILURE MODES OF FRP ILLUSTRATED BY THE ACI 440.2R-08

TABLE 1: STRENGTHENING DESIGN GUIDELINES ADDRESSED BY THE FOUR DESIGN STANDARDS (ACI 440.2R-08, *FIB* BULLETIN 14, HB 305-2008 AND DR AS 5100.8:2016)

Strengthening scheme	ACI 440.2R-08	*fib* Bulletin 14	HB 305-2008	DR AS 5100.8:2016
External bonding (EB)	√	√	√	√
Near surface mounting (NSM)	√	-	-	√
Steel plate	-	-	√	-
Flexural capacity	√	√	√	√
Shear capacity	√	√	-	√
Torsional capacity	-	√	-	√
Axial capacity (Confined members)	√	√	-	√
Prestressed concrete (in flexure)	√	√	-	-
Prestressed FRP	-	√	-	-
Anchorage	√	√	√	√
Elevated Temperature	√	√	-	-
Creep and fatigue rupture	√	√	-	√
Mechanical shear connectors (bolts)	-	-	√	-
Bridge bearing	-	-	-	√
Deck joints	-	-	-	√
Barriers	-	-	-	√
Culverts	-	-	-	√

TABLE 2: DESIGN SAFETY FACTORS PROVIDED BY ACI 440.2R-08, DR AS 5100.8:2016 AND *FIB* BULLETIN 14

Standard		Exposure condition			Application type	
		Interior exposure	Exterior exposure	Aggressive environment	Prefabricated	Wet lay-up
ACI 440.2R-08 and DR AS 5100.8:2016	CFRP	0.95	0.85	0.85	N/A	N/A
	GFRP	0.75	0.65	0.50	N/A	N/A
	AFRP	0.85	0.75	0.70	N/A	N/A
fib Bulletin 14	CFRP	N/A	N/A	N/A	0.83	0.74
	GFRP	N/A	N/A	N/A	0.77	0.67
	AFRP	N/A	N/A	N/A	0.80	0.69

NOTE: For the sake of comparison, the values shown for the *fib* Bulletin 14 are the inverse values of those given in the original table of the standard.

5.2. DESIGN PROCEDURES

Both the ACI 440.2R-08 and *fib* Bulletin 14 explained the steps of external strengthening of flexural members using FRP plates or laminates. In general, the new edition of ACI 440.2R-08 summarized the detailed steps with a given example. The initial considerations given by the standards are the calculation of the strain at the concrete tension face prior to strengthening. The ACI 440.2R-08 assumes that the strain at the initial state of concrete is calculated using elasticity analysis of the member considering dead loads are the only loads carried by the beam (Clause 10.2.3). The *fib* Bulletin 14 states that the initial strain is calculated from the initial moment M_o prior to strengthening (Section 4.2). The design calculation of the flexural strength in ACI 440.2R-08 starts with the initial estimation of the neutral axis depth of the strengthened member and the calculation of the effective strains in the FRP and steel reinforcements. The calculations of the strength of concrete members reinforced with non-prestressed steel follow the simplified stress block concept. The procedures are basically based on the estimations of the neutral axis depth, *c*, using the trial and error methods for checking of the equilibrium of the internal forces (Clause 10.2.10). The controlling mode of failure (either concrete crushing or FRP failure) can be predicted by comparing the computed effective strain in the FRP with the calculated debonding strain. The strain in steel can be computed by the similar triangles of the distribution of the linear strain. After calculating the stresses and strains in the FRP and steel, the initially assumed value of *c* is checked with equation (10-12) in the standard. This procedure is repeated until the values of *c* agree. The cracked section analysis can be used to check the stress level in the steel reinforcement and FRP under service load. After calculating the depth of the neutral axis, the flexural strength of the section can be determined. The service stresses and creep-rupture limits for FRP are also checked in the final stage. The calculation procedures for the NSM and prestressed concrete beams are similar to the procedures mentioned above. However, additional procedures are given for the calculation of bond dependent coefficient for the NSM strengthening. In addition, further checking for the service loads of the steel reinforcement, concrete and FRP are given for the design procedures of prestressed concrete beams.

The design calculation of the shear strengthening consists of four main steps: the properties of materials, level of effective strain in the FRP strips, calculation of the FRP reinforcement contribution to the beam shear strength and the calculation of the overall beam shear strength. Furthermore, the design calculation of the shear reinforcement for the columns involves either the calculation of required area of the FRP wraps (in case the wraps are not installed yet) or the calculation of the contribution of the wraps to the shear strength of the columns. The required design calculation for determining the number of FRP plies for columns under increasing axial load involve the following procedures: determination of the materials properties of FRP, calculation of the required ultimate strength of confined concrete in compression, computation of the confining pressure due to FRP jacket, calculation of the number of required FRP plies and finally verification that the value of the confined concrete maximum axial strain is below 0.01. The simplified curve methodology is used for the case where columns have to carry both axial and bending loads. The procedures start by developing the simplified curves for the unstrengthened columns followed by the simplified curves for the strengthened columns and finally by comparing the required P_U and M_U with the partial interaction diagrams. It should be noted that the aforementioned procedures are used for the non-circular columns. The complete and detailed calculation examples for the design procedures explained above can be found in Part 5 in the ACI 440.2R-08 standard.

The procedures explained by the *fib* Bulletin 14 assume full composite action and the required FRP cross section is calculated from the moment obtained after strengthening (Section 4.7). The ductility should be verified at this stage by checking the limits for the strain and depth of compression zone. The stresses in the concrete need to be computed and the required cross section of FRP should be calculated to fulfill the requirement for stress limits. The width and thickness of FRP are adjusted to fulfill the crack control requirement. The provided FRP cross section should be sufficient to prevent the debonding failure due to the vertical shear cracks. In addition, mechanical anchorage should be provided if the debonding failure either at the FRP ends or at any other location dominates. The necessities for shear strengthening and accidental situation are checked at the final stage of the design. Considerations are given for the design guidelines recommended by the EC2 and MC90 for the design of confined columns. In addition, the capacity of the confined column is calculated based on the equation suggested by Spoelstra and Monti [55]. Considerations need to be given for the type of confinement (full or partial confinement), fibre orientation and column shape (circular or rectangular). Furthermore, different models are described by the standard for enhancing the ductility of confined columns in seismic regions. These models are based on enhancing the plastic hinge of columns by specifying the factor of displacement ductility [56, 57]. The *fib* Bulletin 14 provides equations for the calculation of the effective strain in the FRP for different configurations of shear strengthening (Section 5.1.2). The failure modes associated with each configuration was also mentioned. Based on the different failure modes reported by previous researchers [58], full wrapping of the beams is the most effective method. The standard also suggests attaching FRP strips on the compression zone of beams in the cases where the full wrapping is not possible. In addition, the standard recommends a FRP strips spacing of 0.9d-b_f/2 and d-h_f-b_f/2 for rectangular and T-beams cross sections, respectively, where d, b_f and h_f are the effective depth, width of FRP and thickness of flange (in T-beams), respectively. In addition, equations for the calculation of the torsional forces based on truss mechanism are given in section 5.2 of the standard. Hence, the contribution of the FRP to the torsional strength of the fully wrapped beam can be estimated. The reduction factors shown in Table 2 are used for the calculation of the effective strains of FRP. The factors are 0.8 in the case of FRP rupture failure and 1.80 in the case of FRP debonding.

The HB 305-2008 presented a summarized design procedures for externally bonded beams in section 8.3. The design starts by the calculation of the stress resultants and plate material selection. Different materials can be used at different positions of the plates. This is followed by the calculation of cross sectional area of the plate. Depending on the dominating failure mode, the plate thickness, the need of anchorage and plate length can be calculated. However, the aforementioned guidelines are very general and they depend on the designer who can choose any design approach for the considered element. In addition, the standard did not specify any design guidelines for the confined elements (columns) and prestressed members.

The design principal of DR AS 5100.8:2016 for calculating the flexural capacity of strengthened RC beam, which is similar to the ACI 440.2R-08, is given in equation A6.5.1(1). This equation is based on the total strength provided by the steel reinforcement and FRP. However, the equation uses a multiplier for specifying the location of the resultant compression force (equation A6.5.1(2)). The brief design method for flexure is provided in section A6.6 in the standard. The design starts with the stress block analysis and assessment of the current moment strength. Once the current strength and applied moment are compared and the strengthening is deemed necessary, the section of FRP section and strains limits are determined. The strains in FRP are then computed at the time of concrete crushing and compared with the previously calculated strain limits. The strength of the strengthened section can then be computed and checked with the applied loads. The checks for concrete cover separation mode of failure are performed. This is followed by the final serviceability check. The provisions for the torsion and shear strengthening are explained in section A7 in the standard. Equation A7.2.1 provides the estimate of the shear capacity of the strengthened member, while equation A7.2.2 provides the calculation of FRP contribution to shear capacity of beam. The calculation of effective strain values for full and U-shape wraps is provided in equations A7.2.3(1) and A7.2.3(2), respectively. The torsion capacity is determined in a similar criterion to the flexural and shear strengths (equations A7.3.2 and A7.3.3). The effective strains in completely and U-shape wrapped beams are given in equations A7.3.4(1) and 7.3.4(2), respectively. The axial strength of confined columns under pure axial load can be determined according to equation A8.2.2. However, when the member is subjected to both axial compression and moment, the N-M diagram shall be used with the following requirements: FRP effective does not exceed 0.004 and the failure in compression is the control mode of failure. The modification factors for the non-circular members are presented in equations A8.2.5.2(1) and A8.2.5.2(2). The standard recommends the following anchorage systems for preventing the premature separation of FRP: U-shape wraps, metallic anchors, FRP spikes, and embedment of FRP into grooves made in the members to be strengthened.

5.3.CASE STUDY

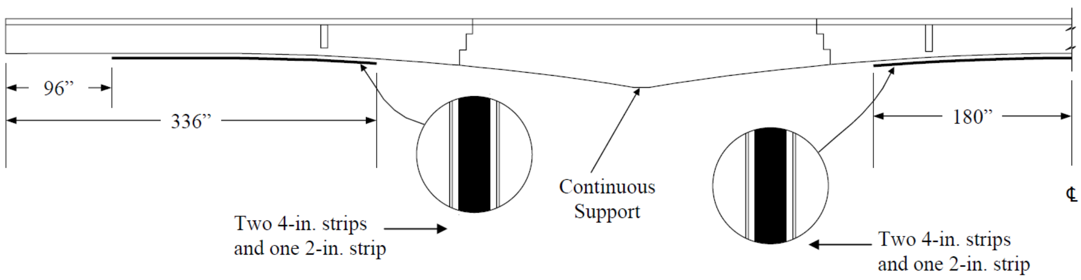
The investigation of behavior of the strengthened existing structures is paramount for collecting data necessary for design calculations and procedures. In addition, monitoring the behavior of the structures carrying service load helps to provide the engineers a better overview of the long term behavior of the strengthened structures. This, in turn, provides more accurate estimation of the necessary safety factors used in design. The performance of existing structures strengthened with FRP and steel plates have been reported in different countries around the world, including US, Japan, Australia, Belgium, UK, Poland, South Africa, Switzerland and France.

A case study is discussed in this section to outline the design considerations and application using FRP on an existing structure. The case study is selected from the Report RP-930-466-2 published by the Alabama Department of Transportation, Alabama, US in 2005 [59]. The bridge is the War Memorial Bridge (Uphapee Creek Bridge) which is located on Alabama State Highway. The bridge was built in 1945 and it has been remarked as structurally deficient. The strengthening system was performed on the bridge three continuous spans using FRP strips.

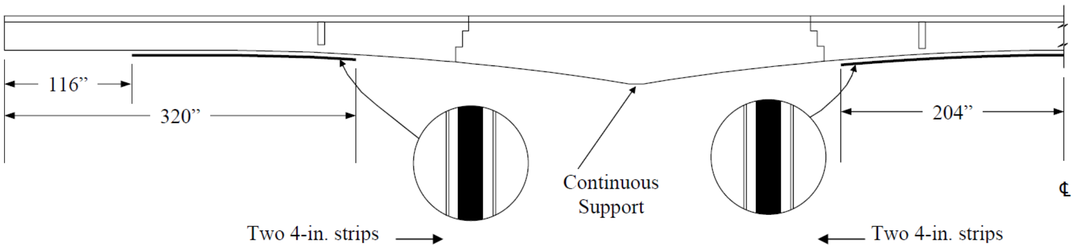
The design and strengthening procedures are explained in detail in the above mentioned report. The strengthening program was performed on the positive bending moment regions of the bridge girders. All the strengthening design methods are based on the recommendations given by ACI 440.2R-02 [60]. The load tests on bridge were performed before and after the strengthening program. The preparation and application of FRP strengthening program are explained in detail in the fourth chapter of the report. The girders surface was prepared by grinders and sand blasting in order to obtain an even concrete surface profile and to remove the dust and other remaining materials that might affect the bonding. The surface preparation was followed by epoxy injection of cracks wider than 0.25 mm. The wet lay-up installation of the FRP laminates started with the application of primer and epoxies in accordance with the manufacturer's recommendations. The strengthening system was inspected upon the completion of the FRP laminates bonding. The inspection was performed according with the guidelines stated in the ACI 440.2R-08. The results of the inspection showed that the overall voids area was less than 0.1% of the inspected area at the FRP/concrete interface. This percentage was considered insignificant in affecting the performance of the strengthening system.

The behavior of the bridge girders was monitored before and after strengthening. A total of four static and dynamic tests were carried out on the bridge girders (Chapter 5). One test was performed before the commencement of strengthening application and the other three tests were performed after the completion of the strengthening. The tests included static and dynamic measurement. Strain gages and deflectometers were used to monitor the bridge girders performance during the tests. The three post strengthening testes were carried out at different time intervals from the strengthening completion. One test was performed immediately after the completion of strengthening and the other two were performed six months later. The different spacing scenarios tests provided by AASHTO [61, 62] for the design trucks were performed. A finite element model for the bridge was also developed using the Automatic Dynamic Incremental Nonlinear Analysis (ADINA) software. The results obtained from the model were compared with the experimental measurements. The detailed description of the model including the assumptions, techniques of the model as well as the data processing can be found in Chapter Six of the report.

The strains in the steel reinforcement and FRP are discussed in the report sections 7.5 and 7.6, respectively. The correlation between the experimental and numerical modeling results generally showed good agreement. Both experiments and numerical model results showed an overall reduction of 5% in steel strains after strengthening. However, the strains obtained from the model were found to be higher than those obtained from experiments by 25% except for one span (Span 10 Girder 3, where the strains were lower by 8%). In addition, the bond was found in a good condition six months after the completion of the strengthening application. The results obtained from this case study showed that the use of FRP for strengthening and retrofitting existing structures is an easy and quick process. However, the researchers reported that based on the site observations during the process of installation and bonding of FRP, the strengthening work is messy and needs proper cleaning. Furthermore, the report shows that performing of the strengthening work while the bridge is under service does not affect the integrity of strengthening system. This finding is of great importance as the disruption of traffic can be avoided during the strengthening application.



(a)



(b)

FIGURE 2: LOCATIONS OF FRP STRIPS ON UPHAPEE CREEK BRIDGE GIRDERS: (A) EXTERIOR GIRDER (B) INTERIOR GIRDER (1 INCH = 25.4 MM)

5.4.FIRE ENDURANCE

The lowest glass transition temperature (T_g) of the components in the strengthening system is considered as the critical temperature of the strengthened structural element. The research on the behavior of strengthened elements under high temperatures is still limited and therefore limited design provisions for the fire endurance is available in the design standards. The loss in the bond between the FRP materials after reaching the glass transition temperature of the epoxy resin is the common failure modes of the FRP strengthened elements. The typical range of the T_g of the resin is usually between 60 to 82 °C for the FRP materials used for strengthening application as specified by the ACI 440.2R-08. The standard recommends that the temperature of the strengthening system should not exceed the value of its (T_g – 15 °C) according to published research works by Luo and Wong [63] and Xian and Karbhari [64]. As stated by the ACI 440.2R-08, reference should be given to the methodologies discussed in ACI 216R [65] for the performance of concrete elements at high temperatures in order to investigate the fire endurance of strengthened elements. The moment resistance of a structural member under elevated temperature can be calculated using the following equation (ACI 440.2R-08, section 9.2.1):

$$R_{n\theta} \geq S_{DL} + S_{LL} \qquad \text{Equation 2}$$

The calculations using the above equation should be based on the reduced properties of the constituent materials of the concrete element during the fire exposure. Moreover, as discussed in the ACI 440.2R-08, section 5.1, the strengthened member should have a reasonable flexural strength at the event of FRP loss, such as the debonding of FRP at high temperatures. The *fib* Bulletin 14 recommends the curing of the adhesives in order to increase its T_g. The standard recommends different evaluation procedures specified by the Eurocode 2 Part 2-2 [66] for the evaluation of the performance of the unstrengthened members (tabulated data, simplified method or general method). In addition, the standard recommends the use of the general method for calculating the flexural strength of elements with fire protection (section 9.3.3). However, the calculation should ignore the contribution of FRP to the flexural strength of the member.

The ACI 440.2R-08 recommends the use of a proper insulation system to enhance the fire endurance of the strengthened member. A number of insulation systems consisting of different materials have been recently investigated, such as the calcium silicate (CS), vermiculite gypsum (VG), vermiculite perlite cement (VP) and vermiculite Cement (VC) [67-75]. It is noteworthy that the standards did not discuss in detail the different types of the insulation systems and no specific insulation system has been recommended for the FRP strengthened elements in particular.

5.5.DISCUSSION AND RECOMMENDATIONS

The two possible solutions for the unsatisfactory performance of existing structures are either demolition of the structure or carrying out a strengthening program. The strengthening and retrofitting of the existing structure is the best option when factors like the cost of materials, labors, disturbing the other facilities and disruption of services are considered. The need for standardizing the design guidelines and procedures for strengthening has become a necessity in recent years for a safer and more economical design. This is also due to the increasing number of strengthening applications and the enormous number of documented research findings on the behavior of strengthened structures. It should be mentioned that the design standards focused mostly on the FRP materials due to its numerous advantages compared to other materials (e.g. steel) with respect to strength to weight ratio, durability and ease of handling. The discussion and comparison of the four design standards (The American ACI440.2R-08 , the European *fib* Bulletin,the Australian HB 305-2008 and the Australian DR AS 5100.8:2016) showed that the latest version of the ACI 440.2R-08 gives the most detailed and comprehensive design procedures compared to other two standards. The ACI 440.2R-08 guidelines are also supported by design examples on different types of structures and strengthening schemes. This improves the practicality of the FRP as a strengthening material and makes it more acceptable in the structural design field. Although the *fib* Bulletin 14, HB 305-2008 and DR AS 5100.8:2016 provide description of the different models proposed for strengthening of the different elements, the standards do not show systematic design procedures for all the conditions and schemes. DR AS 5100.8:2016 listed only the procedures for flexural strengthening in a brief steps form. There were no detailed design calculation examples like those presented by ACI 440.2R-08. In addition, the standard draft did not provide procedures for shear, torsion or column confinement design. When it comes to the type of materials, the HB 305-2008 considered materials other than the FRP such as steel plates. It also discusses the bonding of material using mechanical connectors such as bolts. This gives the designer more options at certain circumstances where the use of epoxy might be limited.

The above discussion of the different design guidelines and procedures show that the research on the behavior of strengthened structures is still needed in order to provide more comprehensive design methodologies that consider all aspects and conditions of structures. This includes the type of structure to be strengthened (i.e. steel or reinforced concrete), strengthening material, and type of loading and strengthening scheme. The future research needs to focus more on the critical temperature at service loads. A clear understanding of the behavior of strengthened individual elements or the whole structure under fire can provide a better prediction of the performance of structure at fire incidents. This is particularly important for strengthening exiting bridges close to bushfire areas. The investigations of epoxies that have relatively high glass transition temperature (T_g) and the different insulation systems are also needed to enhance the strengthened structures fire endurance and delay the bond loss of the strengthening material. Most of research has focused on the strengthening of columns at seismic loads. The design standards need to include effect of the lateral displacement imposed by the seismic or flood loads on other structural elements, such as beams or slabs. In addition, further research is needed to include the design considerations of prestressed concrete with unbounded steel. More experimental data is required for the performance of full wrapped strengthened elements in order to provide an accurate estimation of the reduction safety factors used in FRP shear strengthening calculations. Furthermore, more case studies on strengthened existing structures are needed in order to examine the research findings and different proposed design methodologies. The case studies should provide a complete evaluation of the whole strengthening process as well as the long term behavior of the structures. This can help for a better understanding of the performance of strengthened structures and enable the researchers and engineers to develop more feasible strengthening systems.

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