Australasian advances in steel-concrete composite bridge and building structures

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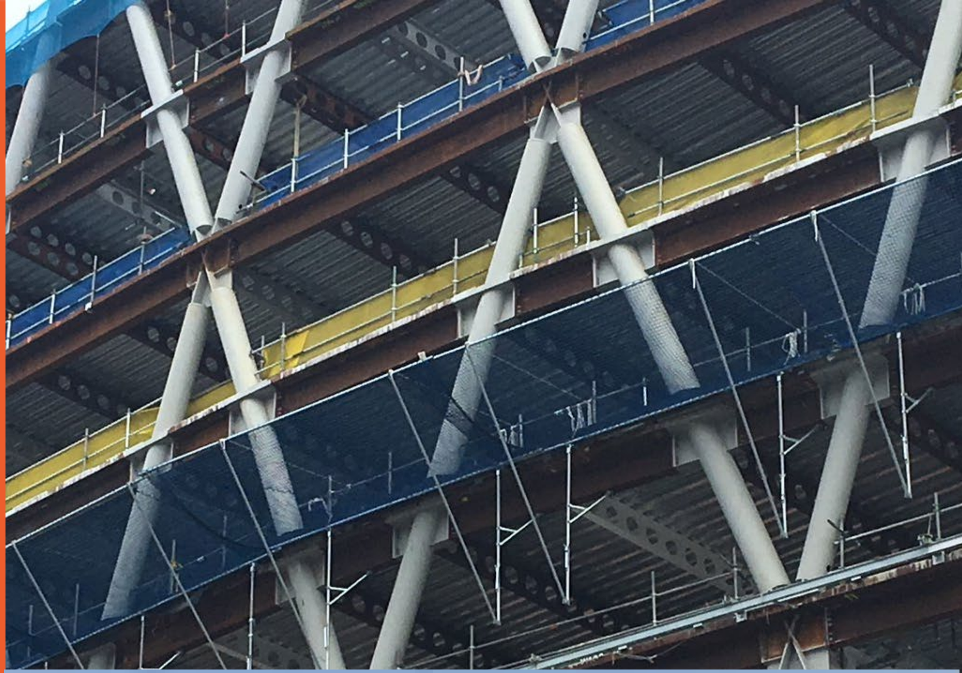

ABSTRACT

This paper will address the Australasian advances in steel-concrete composite bridge and building structures. The paper will firstly provide an overview for the behaviour and design of bridge structures past, present and future. This will then be followed by an overview of the behaviour and design of building structures, past present and future. Over the last decade there have been significant developments on the development of a bridge standard for steel-concrete composite structures, namely AS/NZS 5100 Part 6 and salient elements of recent advances will be highlighted in this paper. In parallel with work being carried out on the development of a bridge standard has been work on the development over the last five years on a building standard for steelconcrete composite structures, namely AS/NZS 2327. Once again, salient features of this standard will be provided in this paper. The paper will conclude with discussions on ongoing and further research that is required in the area of steel-concrete composite structures to deal with the ongoing demands of modern bridge and building structures.

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INTRODUCTION

The concept of steel-concrete composite construction generally produces structural behaviour where the overall response is greater than the sum of the parts. This concept holds true for composite beam behaviour where the introduction of longitudinal shear connection can provide flexural stiffness and strength of a member which is greater than the constituent parts, namely the structural steel section and the reinforced concrete slab. Throughout the latter half of the twentieth century this concept has been further applied to composite steel-concrete columns to produce column stiffnesses and strengths which are greater than the sum of the parts of the steel section and the reinforced concrete elements. These benefits have been further exploited by taking advantage of the confinement effects that steel tubes can provide to interior concrete infill and the subsequent benefits provided by the concrete infill on delaying local buckling of the steel shell. This concept is now so widespread that in the last calendar year, more than 50 % of all tall buildings constructed worldwide, utilized composite frames, typically incorporating concrete filled steel columns (Council of Tall Buildings and Urban Habitat, 2016).

In Australia, builders of the recently completed Perth Tower (the tallest tower in Perth), chose to adopt composite construction throughout the entire structure and a concrete filled steel column solution. In order to secure this type of solution the builder pre-ordered and stored all spirally welded steel tubes in the columns to ensure steel cost fluctuations were minimized and construction costs were able to be controlled (Australian Steel Institute, 2010). The emphasis in Australia has been mainly focussed on construction economy when it relates to steel and steel-concrete composite structures.

INTRODUCTION

CTBUH Year in Review: Tall Trends of 2015, and Forecasts for 2016

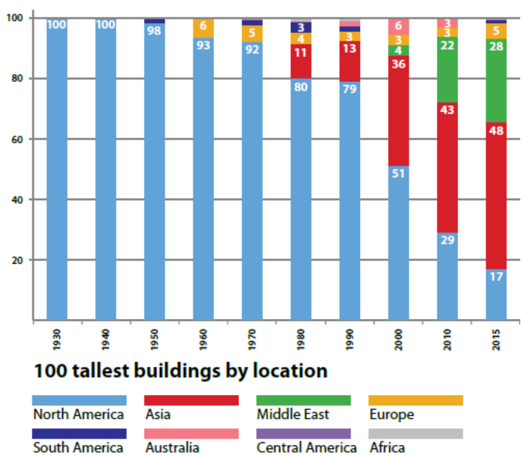
All building data, images and drawings can be found at end
Click on building names to be taken to the Skyscraper Center

The Skyscraper Surge Continues in 2015, The "Year of 100 Supertalls"

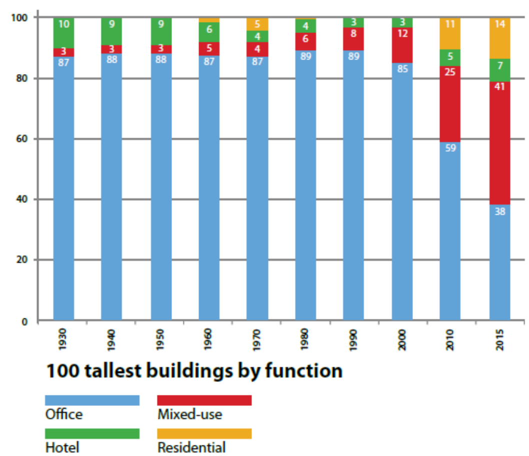
Report by Jason Gabel, CTBUH; Research by Marty Carver and Marshall Gerometta, CTBUH

World's Tallest 100: Analysis

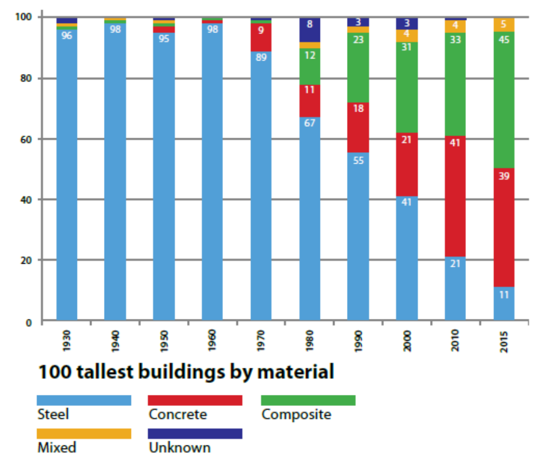
As the graphs below show, we continue to see major shifts towards Asia, mixed-use function, and composite structures.



100 tallest buildings by location



100 tallest buildings by function



100 tallest buildings by material

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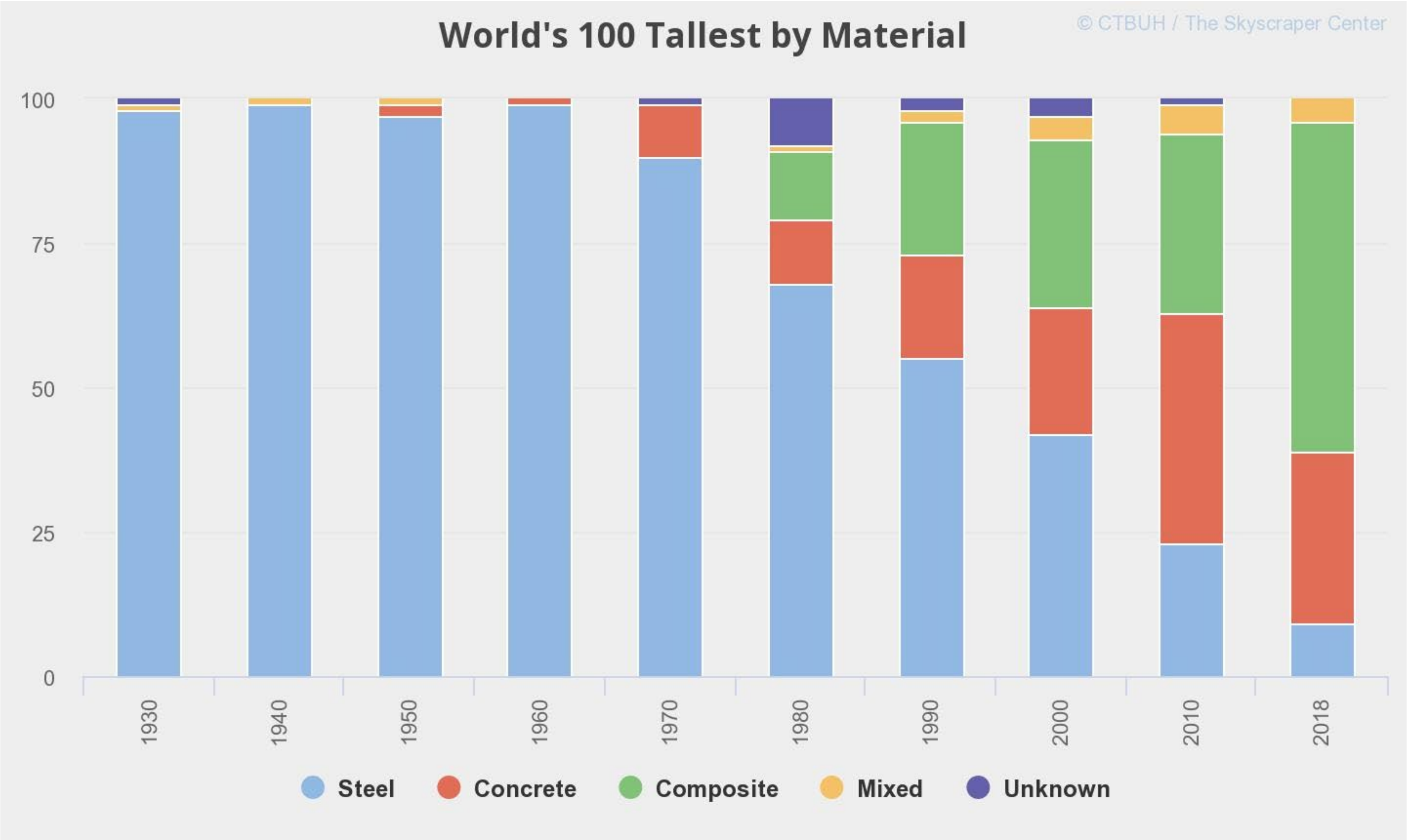

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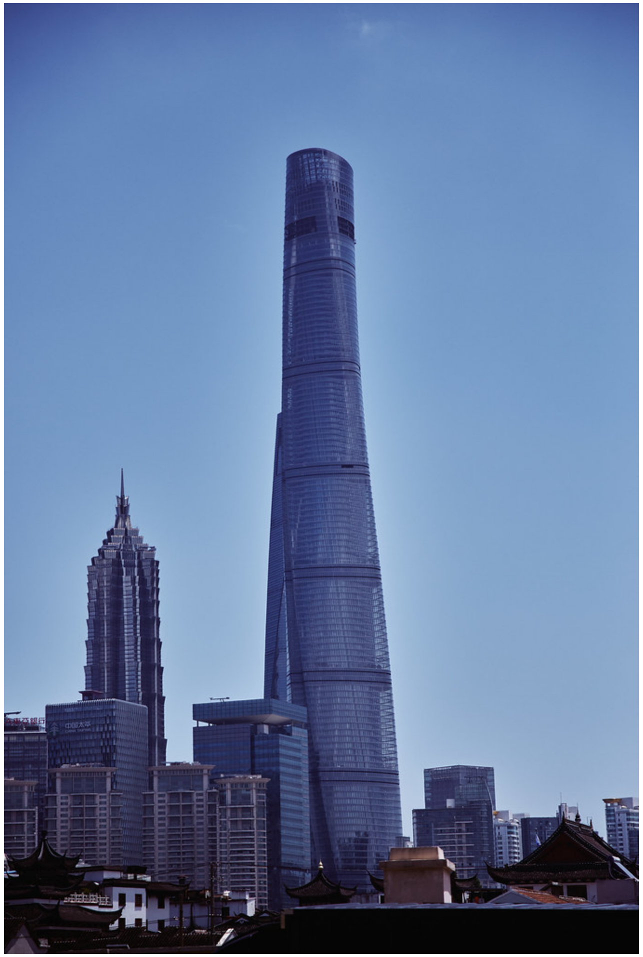

INTRODUCTION

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AUSTRALIAN STANDARD FOR COMPOSITE STEEL-CONCRETE, ASNZS 2327

COMMITTEE BD-032

DR AS/NZS 2327:2016

(Project ID: 100994)

Draft for Public Comment
Australian/New Zealand Standard

LIABLE TO ALTERATION—DO NOT USE AS A STANDARD

BEGINNING DATE FOR COMMENT:	1 November 2016
CLOSING DATE FOR COMMENT:	24 January 2017

Important: The procedure for public comment has changed – please read the instructions on the inside cover of this document.

Composite structures—Composite steel-concrete construction in buildings

(Revision of AS 2327.1—2003)

STANDARDS Australia

STANDARDS NEW ZEALAND
PAEREWA AOTEAROA

DRAFT BUILDING STANDARD FOR COMPOSITE STEEL-CONCRETE, ASNZS 2327

The Australian Standard for composite steel-concrete structures in buildings, AS2327.1 only ever covered the design of simply supported composite beams (Standards Australia, 2003). A major initiative some 5 years ago involved ensuring that all forms of composite systems, including beams, slabs, columns and joints would be covered for design and has resulted in the Australia/New Zealand harmonisation of the standard, ASNZS2327 (Standards Australia/Standards New Zealand 2017a). The standard table of contents is shown below and salient features of the standard will described herein

- SECTION 1 GENERAL
- SECTION 2 DESIGN OF COMPOSITE SLABS
- SECTION 3 DESIGN OF COMPOSITE BEAMS
- SECTION 4 DESIGN OF COMPOSITE COLUMNS
- SECTION 5 DESIGN OF COMPOSITE JOINTS
- SECTION 6 DESIGN OF COMPOSITE FLOOR SYSTEMS
- SECTION 7 SYSTEM DESIGN FOR FIRE RESISTANCE
- SECTION 8 DESIGN FOR EARTHQUAKE
- APPENDICES

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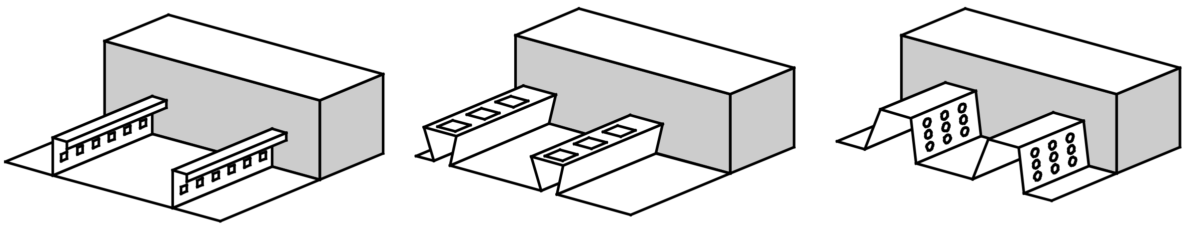
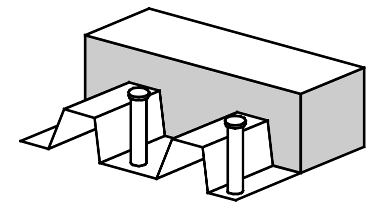
DESIGN OF COMPOSITE SLABS

Section 2 of ASNZS2327 covers the comprehensive design of composite slabs. The intent of this section is to cover the strength and serviceability design of composite slabs utilising metal decking. Issues including flexural strength, longitudinal shear and vertical shear provisions are covered in this section. Concepts of partial interaction are also considered and this section also links quite closely to that being proposed for testing in the Appendices of the standard. Furthermore, post-tensioned concrete construction is also extremely prominent in Australian buildings and recent innovations into post-tensioning concrete slabs with metal decking have been carried out. One of the major issues is the changes that need to be introduced to deal with the presence of the metal decking for serviceability and strength provisions and these will be considered as part of this section, namely the concepts of non-uniform shrinkage (Al Deen et al., 2015).

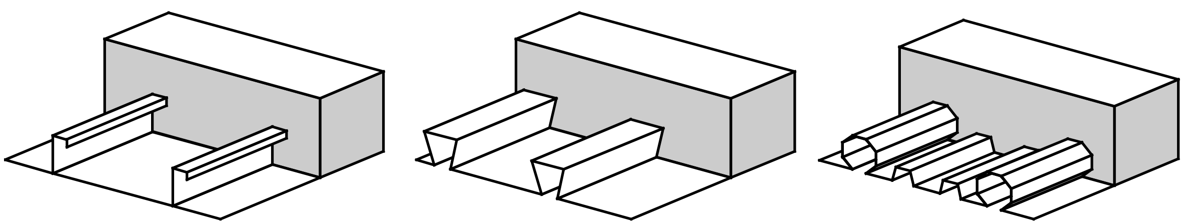
DESIGN OF COMPOSITE SLABS



(a) Mechanical interlock



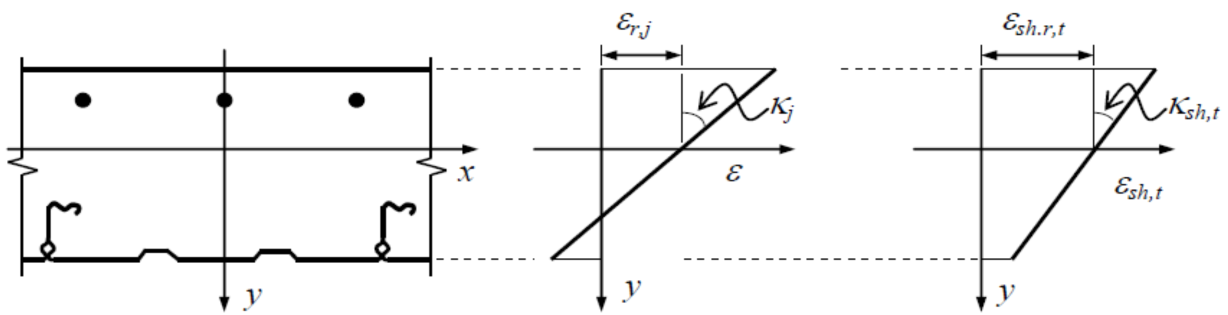
(c) End anchorage by through-deck welded studs



(b) Frictional interlock

DESIGN OF COMPOSITE SLABS

DESIGN OF COMPOSITE SLABS



(a) Typical composite slab cross-section (b) Strain diagram at time t_j (with $j = 0, t$) (c) Shrinkage profile at time t

DESIGN OF COMPOSITE SLABS

2.8.3.4 *Shrinkage deflection*

The deflection component due to shrinkage δ_{cs} shall be determined from a long-term analysis considering only shrinkage effects expressed by the following shrinkage-induced curvature:

$$\kappa_{cs} = (1 - \gamma_{cs})\, \kappa_{cs,cr} + \gamma_{cs}\, \kappa_{cs,uncr} \qquad \ldots 2.8.3.4$$

and

$$\gamma_{cs} = \left(\frac{M_{cr}}{M_s^*}\right)^2 \leq 1$$

where

$\kappa_{cs,uncr}$ = curvature produced by shrinkage over an uncracked section based on a long-term analysis and considering the effective modulus of elasticity $E_{ef.cs}$ for the concrete component

$\kappa_{cs,cr}$ = curvature produced by shrinkage over a cracked section based on a long-term analysis and considering the effective modulus of elasticity $E_{ef.cs}$ for the concrete component

$$E_{ef.cs} = \frac{E_c}{1 + 0.55\varphi_{cc}}$$

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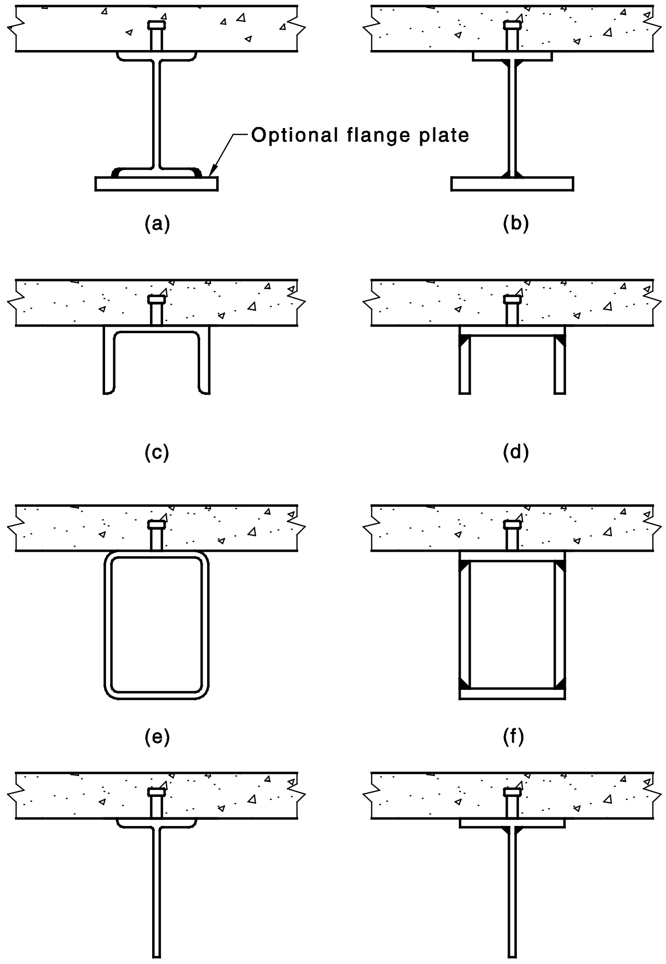
DESIGN OF COMPOSITE BEAMS

Section 3 of ASNZS2327 covers the comprehensive design of composite steel-concrete beams. This section covers the design of composite beams, considering flexural strength, shear strength and combined actions as well as serviceability provisions. Partial shear connection approaches are also highlighted for the design of simply supported and continuous beams. This section also considers the design of composite beams using hollow core slabs as illustrated, (Uy and Bradford, 2007).

DESIGN OF COMPOSITE BEAMS



DESIGN OF COMPOSITE BEAMS

DESIGN OF COMPOSITE BEAMS

3.5.5 Design vertical shear capacity

The shear strength of a composite section consists of the individual contributions of the shear capacity of the steel beam alone, $V_{pl,Rd}$, and the concrete slab, V_{slab} as follows:

$$V_{comp} = V_{pl,Rd} + V_{slab} \qquad \ldots 3.5.5(1)$$

where

$V_{pl,Rd}$ = shear strength of the steel beam alone and calculated according to AS 4100 or NZS 3404 for compact steel sections

$= \phi V_{uw}$

V_{slab} = accounts for the effects of both the shear strength of concrete slab and the shear connection between the slab and the beam

Thus, the shear strength of the concrete slab is proposed to be calculated by:

$$V_{slab} = \phi_s f(\lambda_{sd})(b_f D_{slab})^{0.7}\sqrt{f'_c} \qquad \ldots 3.5.5(2)$$

where

ϕ = capacity factor, see Table 1.4.3 for shear

$f(\lambda_{sd})$ = function of the slab slenderness ratio

D_{slab} = Depth of slab which shall be taken as: h for solid slabs, re-entrant and clipped pan profiles; or h_c for open trough profiles (see Figure 2.1.1 for definitions).

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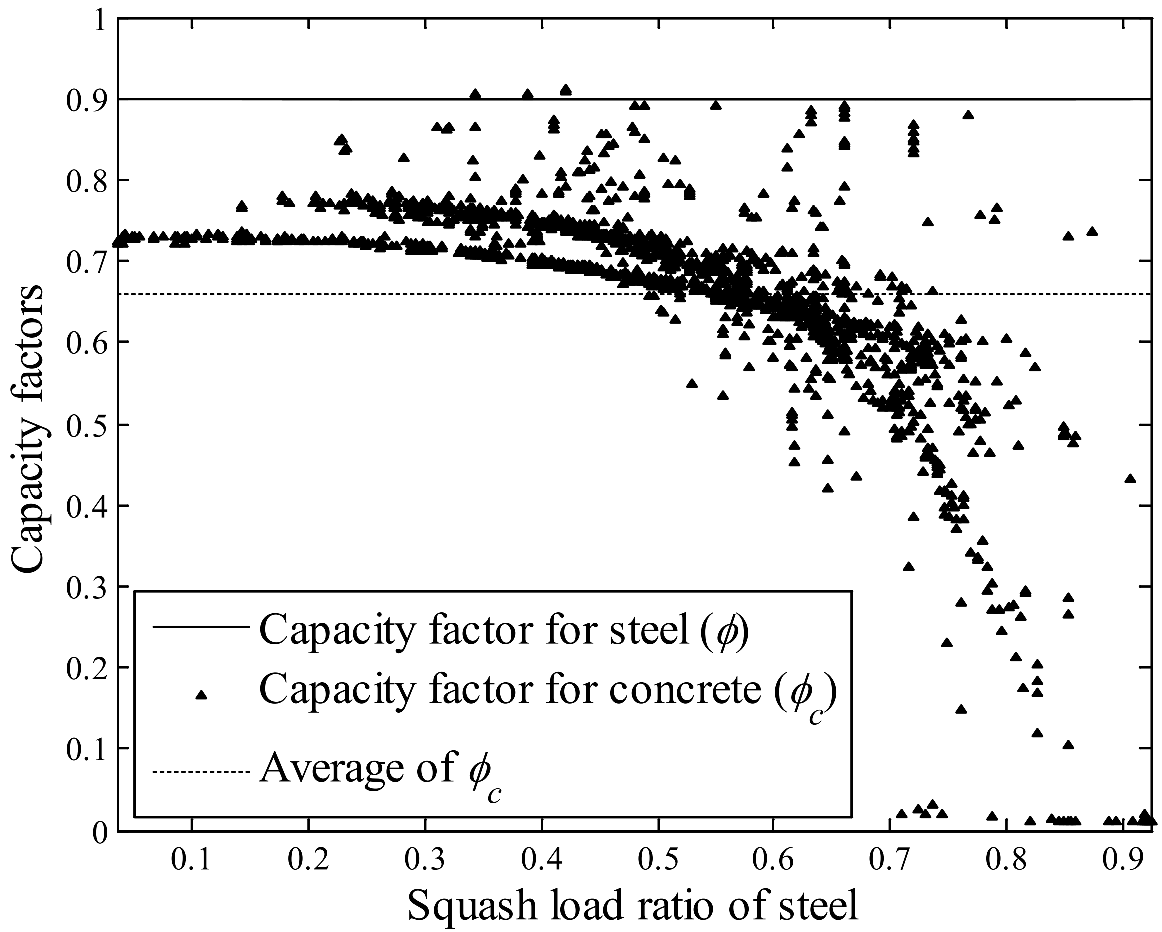
DESIGN OF COMPOSITE COLUMNS

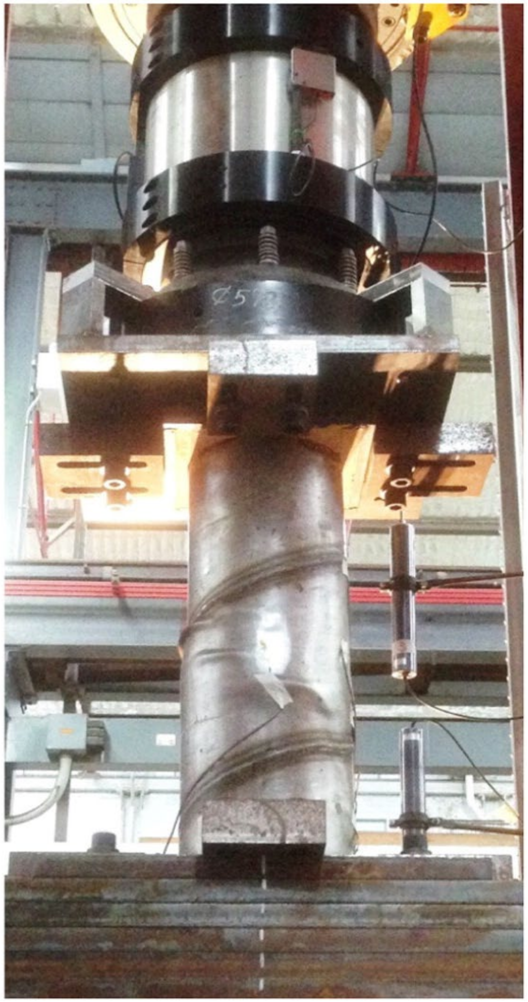
This section covers the design of composite columns and closely follows the approach in the AS/NZS 5100 Part 6 (Standards Australia/Standards New Zealand, 2017b). The design of composite columns for strength, stability incorporating axial force, uniaxial and biaxial bending is considered. In particular, the important effects of confinement are covered by this section. Furthermore, the capacity factor for concrete in compression is proposed to be 0.65 based on reliability analyses using the design assisted by testing method provided in EN 1990 Annex D.8 (European Committee for Standardization, 2002). The reliability analyses were carried out for 1583 CFST columns included in Tao et al.'s database (Tao et al., 2008).

DESIGN OF COMPOSITE COLUMNS



DESIGN OF COMPOSITE COLUMNS
Perth Tower, 2012

DESIGN OF COMPOSITE COLUMNS



DESIGN OF COMPOSITE COLUMNS

Table 3 Codes of practice models for *EI*

Ref.	*EI* specification
AS 5100.6 (2004)	$(EI)_e = \phi EI_s + \phi EI_r + \phi_c E_c I_c$ $E_c = 5050\sqrt{f'_c}$ $E_s = 200{,}000$ MPa $\phi = 1.0$ for composite members $\phi_c = 1.0$ for composite members
AS/NZS 2327 (Draft 2015)	$(EI)_e = E_s I_s + E_s I_{sr} + 0.6 E_c I_c$ $E_c = w^{1.5}(0.024\sqrt{f_{cmi}} + 0.12)$ $E_s = 200{,}000$ MPa
Eurocode 4 (2004)	$(EI)_e = E_a I_a + E_s I_s + 0.6 E_{cm} I_c$ $E_{cm} = 22{,}000[(f_{cm}/10]^{0.3}$ $E_s = 210{,}000$ MPa
AISC (2010)	$EI = E_s I_s + E_s I_{sr} + C_3 E_c I_c$ for CFSTCs $E_c = 0.043 w_c^{1.5}\sqrt{f'_c}$ $E_s = 200{,}000$ MPa $C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.9$ $EI = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c$ for CESCs $C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.3$
ACI 318 (2010)	$EI = \frac{0.2 E_c I_g + E_s I_{se}}{1 + \beta_d}$ $E_c = 4734\sqrt{f'_c}$ $E_s = 199{,}948 MPa$ $\beta_d = 0$ for short-term loads
AIJ (1997)	$K_e = E_s I_s + 0.2 E_c I_c$ $E_c = 2100\sqrt{f'_c/19.6}$ $E_s = 205{,}800 MPa$

Table 7 Reliability indices of composite columns against serviceability limit state

$\beta_{seviceability}$	RCFSTC	CCFSTC	Full. Encased	Par. Encased
AS 5100.6 (2004)	1.05	0.54	1.54	> 3.00
AS/NZS 2327 (Draft 2015)	1.36	0.95	> 3.00	> 3.00
Eurocode 4 (2004)	1.33	0.91	> 3.00	> 3.00
AISC (2010)	1.22	0.79	> 3.00	> 3.00
AIJ (1997)	1.69	1.33	> 3.00	> 3.00
ACI 318 (2010)	1.72	1.36	> 3.00	> 3.00

DESIGN OF COMPOSITE COLUMNS

For RCFSTCs, CCFSTCs, and CESCs, the AIJ (1997) and ACI 318 (2010) show the lowest standard deviations, making them more reliable predictors for deflection. Both AIJ (1997) and ACI (2010) use a capacity factor of 0.2 in the *EI* equations, which is far lower than the other codes. The AS5100.6 (2004) uses a factor of 1.0 and AS/NZS 2327 (Draft 2015) uses a factor of 0.6. From these results, a capacity factor of 0.2 provides the most reliable deflection results.

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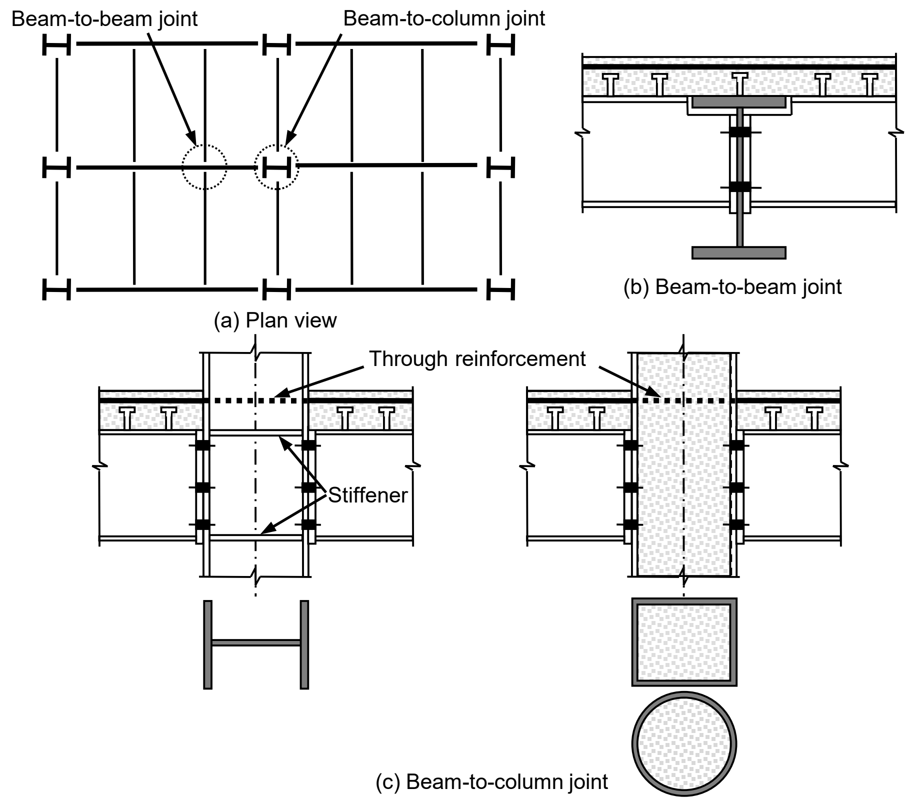
DESIGN OF COMPOSITE JOINTS

This standard has also involved in the development of a section for strength and serviceability design of semi-rigid joints including beam-to-beam and beam-to-column joints as shown in Figure 9. For beam-to-column joints, the column could be either open sections with/without stiffeners or hollow sections with/without infilled concrete. The design of joints to hollow section columns is based on the stiffness model of Thai and Uy (2016) which was calibrated with experimental results of 44 available tests on bolted endplate beam-to-CFST column joints.

DESIGN OF COMPOSITE JOINTS



DESIGN OF COMPOSITE JOINTS

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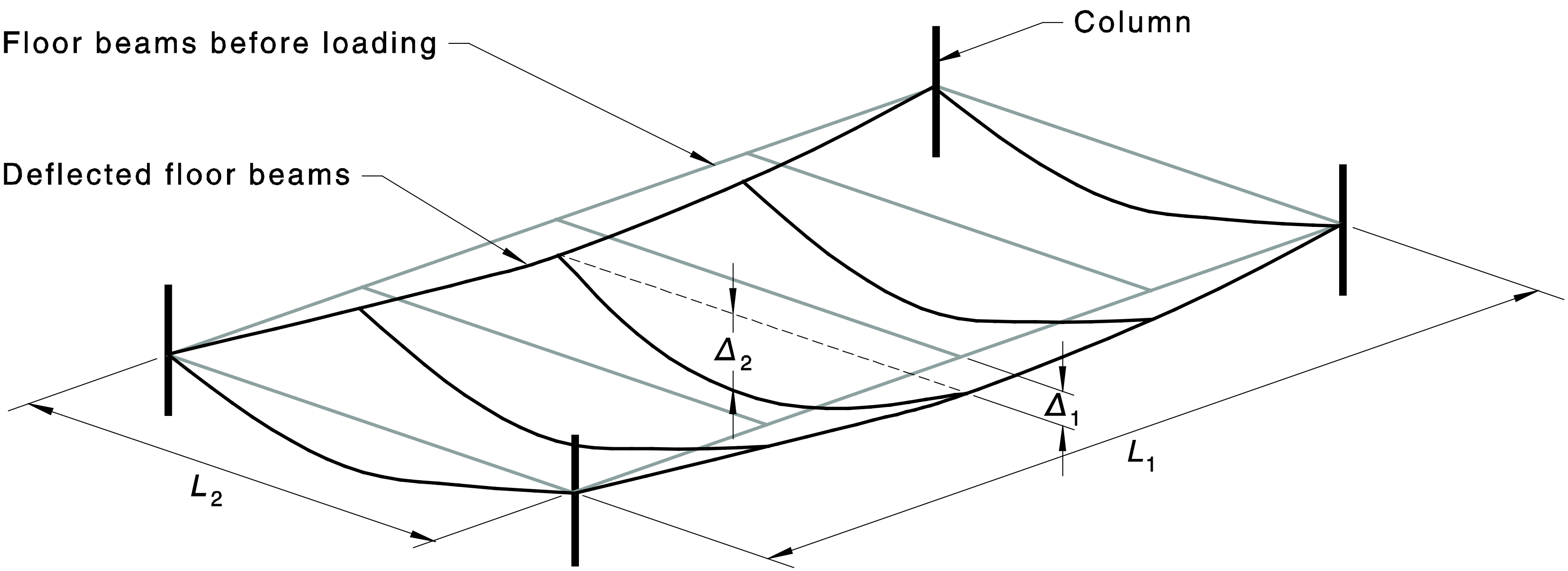
DESIGN OF COMPOSITE FLOOR SYSTEMS

The intent of this section is to address system behaviour particularly for deflections and vibrations for panels. This will then give designers the ability to take into account the beneficial effects of system behaviour in addressing these important serviceability provisions which sometimes penalize steel frame structures from a design perspective (Steel Construction Institute, 2012).

DESIGN OF COMPOSITE FLOOR SYSTEMS



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SYSTEM DESIGN FOR FIRE RESISTANCE

This section is also intended to give guidance on design for fire using a system based approach, which acknowledges that for indeterminate systems there is a significant degree of redundancy that provides additional structural capacity within a fire that is unable to be addressed considering single elements within a building. State of the art approaches for dealing with this will be provided herein (Steel Construction Institution, 2006 and Abu et al., 2011).

DESIGN FOR FIRE RESISTANCE

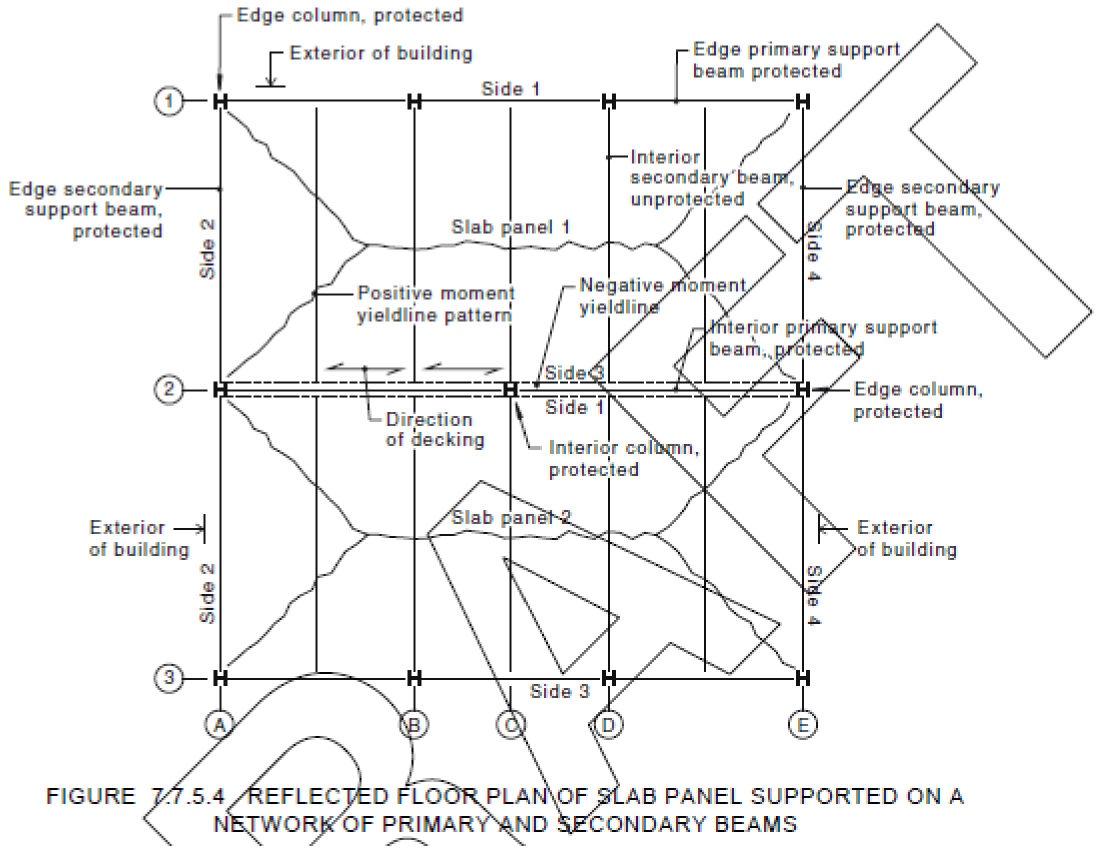
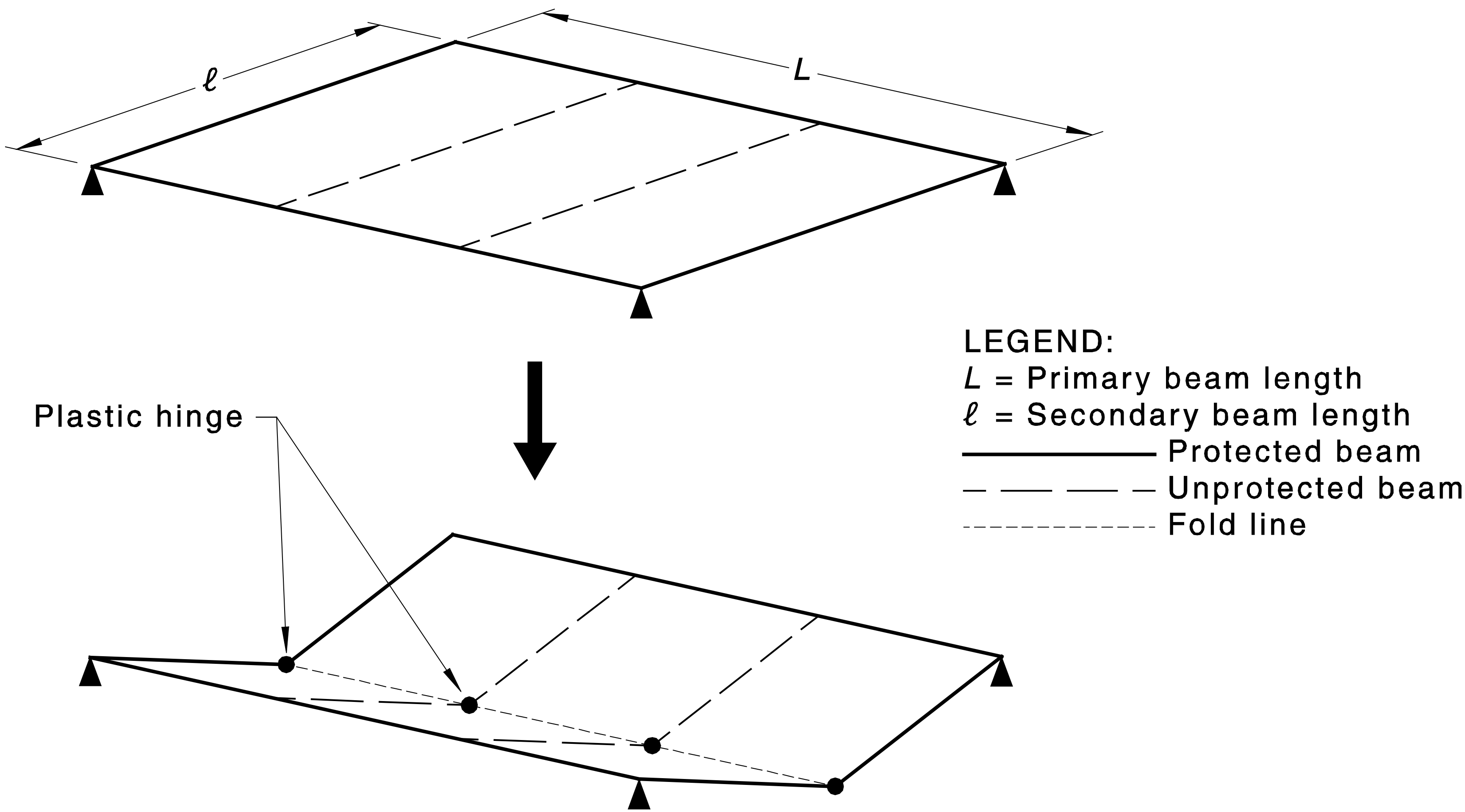


FIGURE 7.7.5.4 REFLECTED FLOOR PLAN OF SLAB PANEL SUPPORTED ON A NETWORK OF PRIMARY AND SECONDARY BEAMS

SYSTEM DESIGN FOR FIRE RESISTANCE



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DESIGN FOR EARTHQUAKE

This section covers the design for earthquake of composite frame systems. One of the major elements of the section is the specific guidance provided to Australian and New Zealand designers in accordance with AS1170.4 and NZS1170.5 (Standards Australia 2004 and Standards New Zealand 2007).

DESIGN FOR EARTHQUAKE

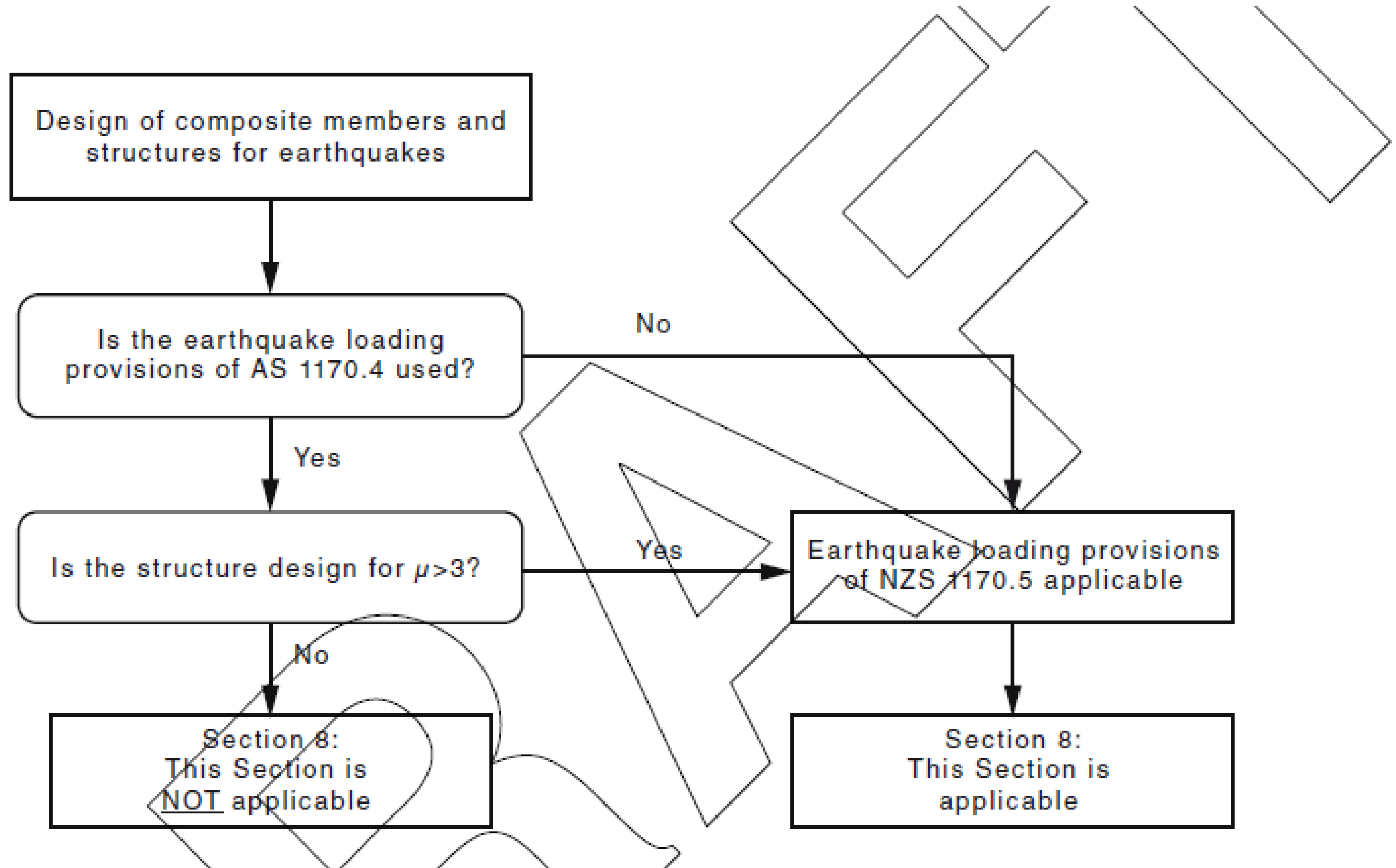

FIGURE 8.1 FLOWCHART FOR DESIGN OF COMPOSITE MEMBERS AND STRUCTURES FOR EARTHQUAKES

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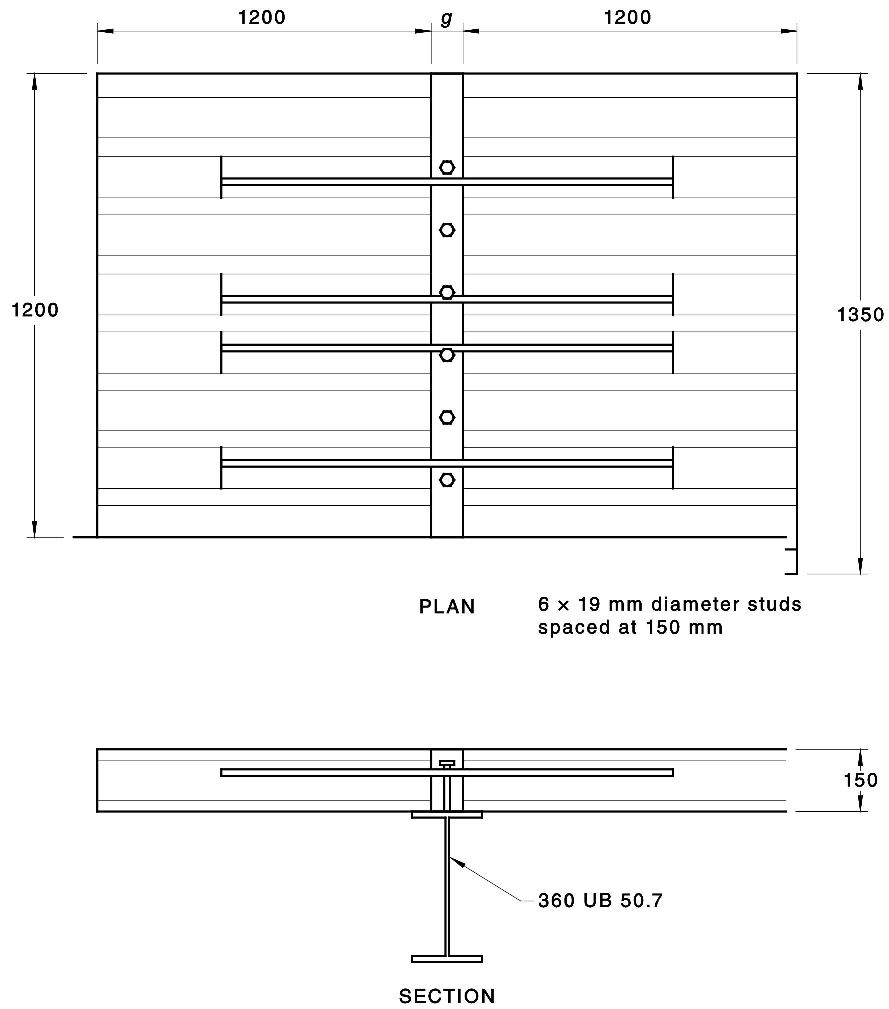
APPENDICES

This section of the proposed standard is meant to provide standard test methods for a number of specific issues which are covered. Push test methods for establishing the strength, stiffness and ductility of shear connectors will be outlined in this section as will test methods for establishing the strength characteristics of composite slabs incorporating profiled steel sheeting. In addition, provisions for evaluating design resistance from tests will also be presented. Finally, ASNZS 2327 will be one of the first international composite design standards to present provisions for beams with both regular and isolated web-openings, thereby supporting the use of long-span cellular beams (Steel Construction Institution, 2011).

APPENDICES



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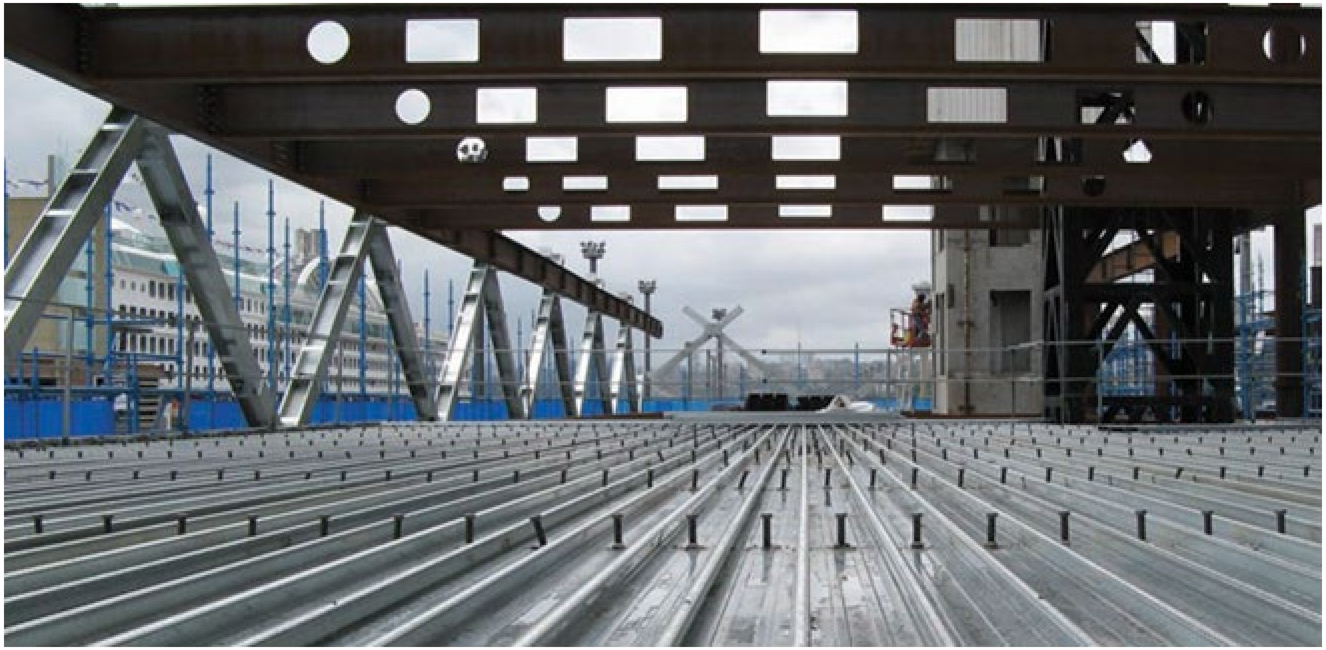
1350
PLAN
6 x 19 mm diameter studs
spaced at 150 mm
150
360 UB 50.7
SECTION

APPENDICES



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CONCLUSIONS AND FURTHER RESEARCH

Whilst there is ongoing research into structural steel and some of the technical issues associated with materials and systems, it is felt that further research will be punctuated by approaches that provide paradigm shifts in the design of steel and steel-concrete composite building structures. Some of the more prominent issues that will promote these paradigm shifts include precast and prefabricated construction, deconstructability and new and higher performance materials.

In particular steel and steel-concrete composite framed buildings have the ability to be designed and constructed with deconstruction in mind. Advanced economies around the world are now looking toward reuse as a potential for addressing the issue of shortages in natural resources in future. Composite action has the ability to reduce steel usage and deconstruction has the ability to provide reuse options. Reduce and reuse strategies are far superior to recycling options and have the ability to provide much greater benefits to society from the perspectives of sustainability.

Minor revisions are being planned and a revised amendment is planned for publication in 2020.

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ACKNOWLEDGEMENTS

The authors would like to thank the members of the BD-32 Committee for all their hard work to date; in particular, the Standards Australia Project Manager.

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THE AUSTRALASIAN COMPOSITE BRIDGE STANDARD, AS/NZS 5100 PART 6, STEEL AND COMPOSITE CONSTRUCTION

THE UNIVERSITY OF SYDNEY

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Introduction

This lecture will highlight the development of the new Australasian Standard on Bridge Design, Steel and Composite Construction, AS/NZS 5100:6 which was released in 2017. Harmonisation across the Tasman Sea has been gathering pace ever since the development of the AS/NZS 4600 (Standards Australia, 2005) in cold formed steel structures was released and there are also moves afoot to harmonise the existing AS4100 and NZS3404 into an Australian/New Zealand standard on steel structures, (Standards Australia, 2012 and Standards New Zealand, 1997).

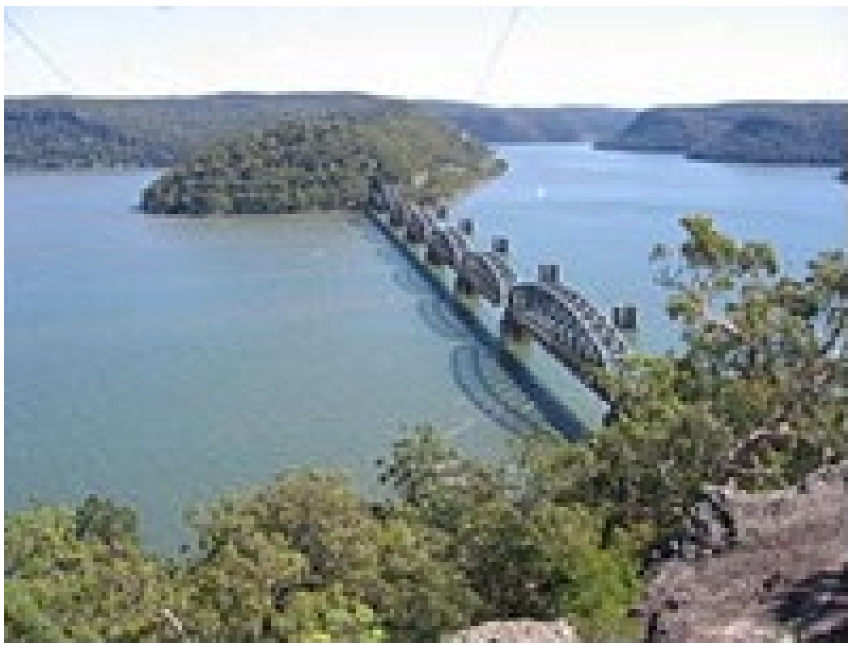
Introduction

This lecture highlights the structure of the standard and includes salient changes from the earlier version, including in the area of high strength concrete, high strength steel and shear connectors. Furthermore, the issue of imported steel and the reliability studies carried out to determine their appropriateness will also be highlighted in this paper. Many of the changes also draw heavily on overseas European and American Standards, (British Standards Institution, 2005 and American Institution of Steel Construction, 2010).

Sydney Harbour Bridge, Sydney

Te-Rewa-Rewa, New Plymouth, New Zealand

Hawkesbury River Rail Crossing, Sydney



The 123 year old rail crossing of the Hawkesbury River north of Sydney and the 80 year old Sydney Harbour Bridge. The Hawkesbury River rail bridge was designed and built by the Union Bridge Company from New York, USA.

Sydney Harbour Bridge, Sydney

The Sydney Harbour Bridge completed in 1932 was based on a general design by the NSW Department of Public Works but heavily based on New York's Hell Gate Bridge and with detailed design by Dorman Long and Co, Middlesborough, UK through Sir Ralph Freeman and Sir Douglas Fox.

Hawkesbury River Road Bridge, Sydney

The Hawkesbury River Road Bridge completed in 1977 was designed and constructed as a steel box girder bridge with shear connection making the concrete deck composite through the top flange.

George’s River Bridge, Sydney

A major composite bridge was built over the George’s River in Sydney in 1987. This was a parallel bridge to the 1923 Pratt Truss steel bridge which was completed in 1923. The new bridge consisted on eight 70 m spans involving three steel box girders utilising composite action with the concrete deck.

Roberts Road Bridge, Sydney

A major composite-steel concrete bridge was built in Sydney in 2000 at Roberts Road crossing the Hume Highway and linking northern and southern arterial roads of Sydney. The superstructure of this bridge comprised four steel trough girders supporting a concrete deck across six continuous spans ranging from 25-40 metres. The sections adopted 350 MPa (N/mm^2) steel plate with 1400 mm depth and 2250 width sections.

Clifton Hill Bridge, Melbourne

In addition to some of the more obvious challenges, some other engineering challenges in the bridge design realm, will be in the area of urban design. Architects are finding increasing involvement in the urban design of bridges. Architectural involvement then poses unique challenges for structural engineers which need solutions to be available. A recent bridge designed in Clifton Hill, Melbourne involved significant architectural involvement and penetrations were required to achieve the architectural objective. This structure required significant finite element analysis carried out to justify the designs and this will be a continuing trend in future bridge designs.

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Structure of the Standard

The current structure of the proposed Australian Standard AS/NZS: 5100 Part 6 is given below. Sections 1, 2, 3 and 4 relate to introductory materials such as Scope and General, Materials, General Design Requirements and Methods of Structural Analysis.

Sections 5 and 6 covers the design of steel and composite beams respectively and this is where the introduction of high strength steel and concrete is a new development as will be highlighted later in this paper. Section 7 covers the design of box girders where the introduction of high strength steel and concrete also have some influence on the new revisions, whilst Section 8 deals with transverse members and restraints.

Structure of the Standard

Sections 9, 10 and 11 covers the design of axial tension, axial compression and combined actions of elements. All these sections are affected by the introduction of high strength concrete and steel. The remainder of the standard is devoted to the design of connections, fatigue and brittle fracture, testing of structures or elements and numerous appendices.

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AS/NZS: 5100 PART 6, Steel and Composite Construction

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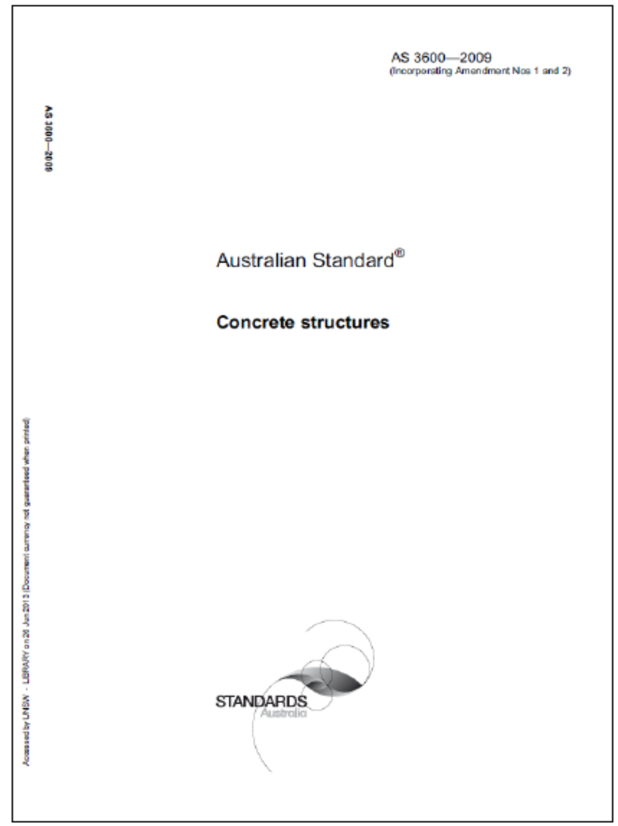
High Strength Concrete

The changes to AS/NZS 5100: Part 6 are in part as a result of the changes to the Australian Standard for Concrete Structures AS3600. This standard has had three versions during the last two decades, AS3600-1994 (Standards Australia, 1994), AS3600-2001 (Standards Australia, 2001) and AS3600-2009 (Standards Australia, 2009) as illustrated in Figure 3. The major changes in these versions were the gradual increase in concrete compressive strengths from 50, to 65 and now 100 MPa (N/mm^2). The steel reinforcing yield strengths have also gradually increased up to 500 MPa (N/mm^2) in the most recent version.


AS 3600—2009
(Incorporating Amendment Nos 1 and 2)

Australian Standard®

Concrete structures

STANDARDS Australia

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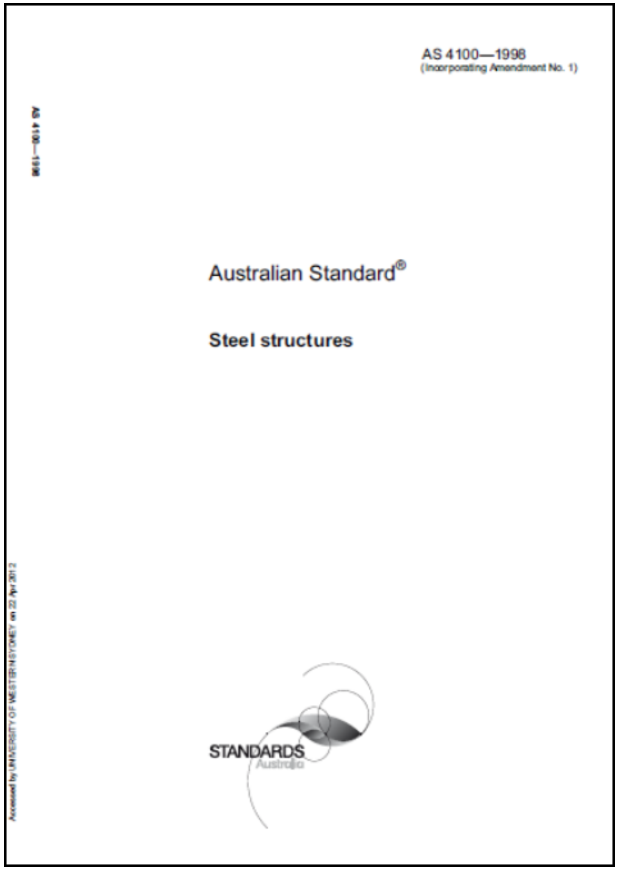
High Strength Steel

AS/NZS: 5100 Part 6 has also required changes as a result of changes to AS4100. The standard for steel structures AS4100 also had two versions during the last two decades, AS4100-1998 and AS4100 Ammendment 1 – 2012, (Standards Australia, 1998 and Standards Australia, 2012) as illustrated in Figure 4. A major change introduced with the new amendment was the introduction of high strength steel up to 690 MPa (N/mm^2).



High Strength Steel Reductions

Clause from proposed standard AS/NZS 5100:6 on reductions for high strength steel

6.2.3 Reduction for use of high strength steel

When using high strength steel, a reduction in the bending capacity to allow partial depth yielding of the beam needs to be considered according to Figure 6.2.3.

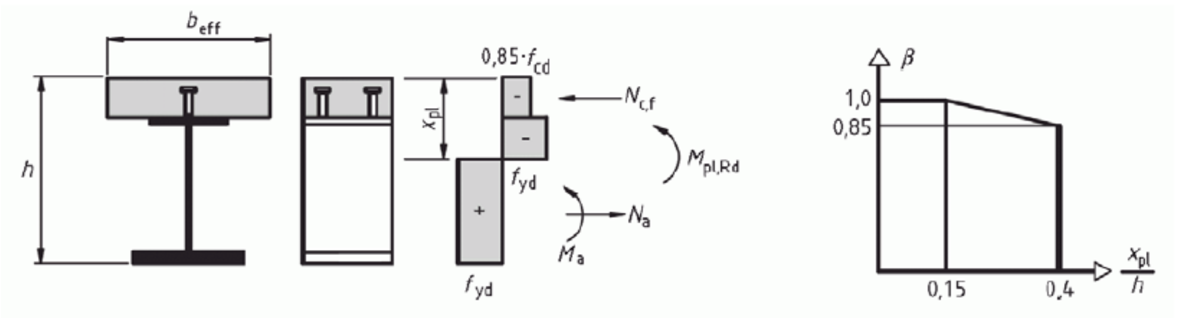


FIGURE 6.2.3 REDUCTION FACTOR β for $M_{pl,Rd}$

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Shear Connectors

The most common form of shear connector in composite construction is the headed stud. Following the introduction of higher gradesof concrete, a review of recent international research resulted in historical push data to be expanded to include 113 results. A reliability study by Hicks and Jones (2013) demonstrated that the following design equations can be safely used to include stud connectors embedded in concrete with a characteristic compressive strength not greater than 80 MPa (where $\phi = 0.8$):

$$f_{ks} = 0.70 d_{bs}^{\ 2} f_{uc}$$

or:

$$f_{ks} = 0.28 d_{bs}^{\ 2} \sqrt{f'_{cy} E_c}$$

whichever is smaller.

where d_{bs} is the nominal shank of a shear stud, but 16 mm $\leqslant d_{bs} \leqslant$ 25 mm; f_{uc} is the ultimate tensile strength of the stud material, but not greater than 500 MPa; f'_{cy} is the characteristic strength of the concrete at the age considered, but 16 MPa $\leqslant f'_{cy} \leqslant$ 80 MPa; E_c is the modulus of elasticity of concrete at the age being considered, and may be taken as $E_c = \rho^{1.5}\left(0.043\sqrt{f_{cmi}}\right)$ for $f_{cmi} \leqslant$ 40 MPa or $E_c = \rho^{1.5}\left(0.024\sqrt{f_{cmi}} + 0.12\right)$ for $f_{cmi} >$ 40 MPa.

The other major innovation is the introduction of the rules for horizontally lying studs in Clause 6.7.4.8 shown hereafter .

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Horizontal lying studs

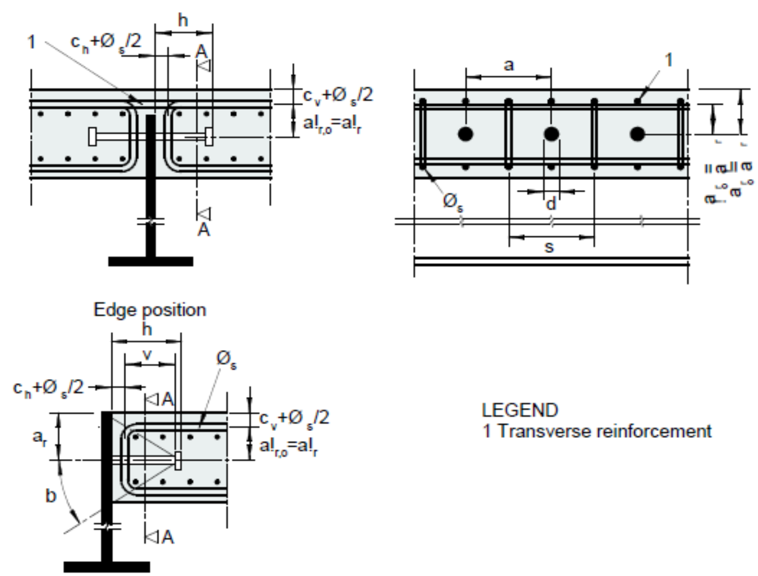

FIGURE 6.7.4.8(B) POSITION AND GEOMETRICAL PARAMETRES OF SHEAR CONNECTORS WITH HORIZONTALLY ARRANGED STUDS

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Reliability of Steels

This section briefly introduces the statistical rationale based on a rigorous reliability analysis for conformity assessment rules for overseas steels. In order to try and allow for the smoother introduction of overseas steel into the Australian/New Zealand market, a reliability study was carried out by Kang, Hicks and Uy (2013) dealing with the manufacturing and material strength tolerances of overseas steels. Table 1 highlights the different manufacturing tolerances considered and the various overseas standards considered, including European, Korean, Japanese, American and Australian/New Zealand tolerances, (British Standards Institution, 1993; Korean Standards Association, 2007; Japanese Standards Association, 2005a,b; ASTM 2012, Standards Australia, 2004 and Standards New Zealand, 1996). Also for material strength variations although a value of 7% is often used internationally (Joint Committee on Structural Safety, 2001), the coefficient of variation for the yield strength of steel was conservatively taken to be 10%, which is consistent with the value used in the original Australian Standard calibration conducted by Pham et al. (1986).

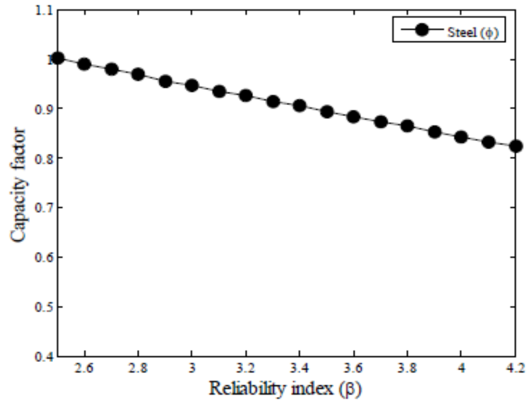
Reliability of Steels

The capacity reduction factor for the design of I-beams was calibrated based on the experimental data provided by Byfield and Nethercot (1998). This data included 32 test results on laterally restrained I-beams, which are all compact based on the plate element slenderness limits given in AS 5100.6 (Standards Australia 2004). Two section types 203x102x23UB and 152x152x30UC with a steel grade FE430A are considered in the data. For the calibration, the following assumptions are made: (1) the mean measured values of fillet radius are not reported in Byfield and Nethercot, and the nominal values are used instead; (2) compact section capacity is calculated based on AS 5100.6, and member capacity is not considered. The calibration was conducted using the statistical method given in EN 1990 Annex D.8 (European Committee for Standardization 2002).

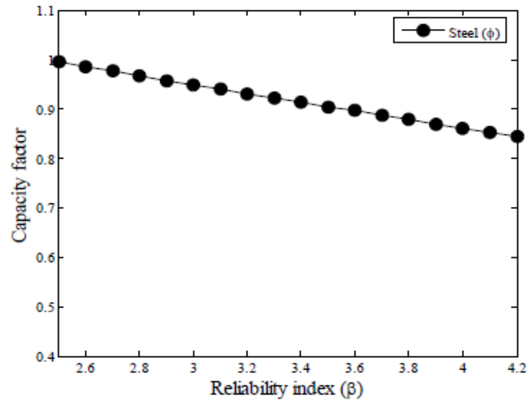
Manufacturing tolerances

<table>
<tr><th>Parameter</th><th colspan="2">EN10034: 1993
KS D 3502: 2007</th><th>JIS G 3192: 2005
JIS A 5526: 2005</th><th colspan="2">ASTM A 6/A 6M - 07</th><th colspan="2">AS 5100.6
AS/NZS 1365:1996</th></tr>
<tr><td>Depth (h) (mm)</td><td colspan="2">-2</td><td>-2</td><td colspan="2">-3</td><td colspan="2">-h/50</td></tr>
<tr><td>Width (b) (mm)</td><td>$b \leq 110$
$110 < b \leq 210$</td><td>-1
-2</td><td>-2.5</td><td colspan="2">-5</td><td colspan="2">-b/100</td></tr>
<tr><td>Web thickness (t_w) (mm)</td><td>$t_w < 7$
$7 \leq t_w < 10$</td><td>-0.7
-1</td><td>-0.7</td><td>$t_w < 7$
$7 \leq t_w < 10$</td><td>-0.7
-1</td><td rowspan="2">$4.5 < t \leq 6$
$6 < t \leq 10$</td><td rowspan="2">-0.85
-0.9</td></tr>
<tr><td>Flange thickness (t_f) (mm)</td><td colspan="2">-1</td><td>-1</td><td colspan="2">-1</td></tr>
</table>

Capacity Reduction Factor

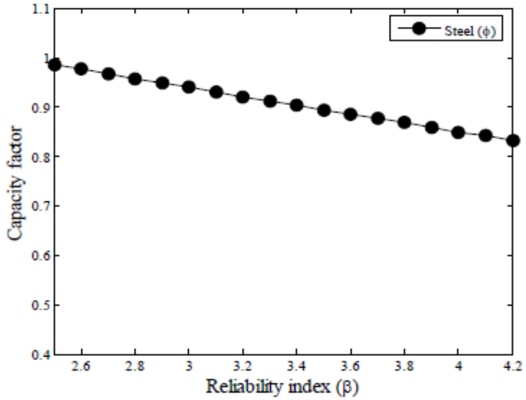


(a) ϕ*=0.94 at* β *= 3.04*

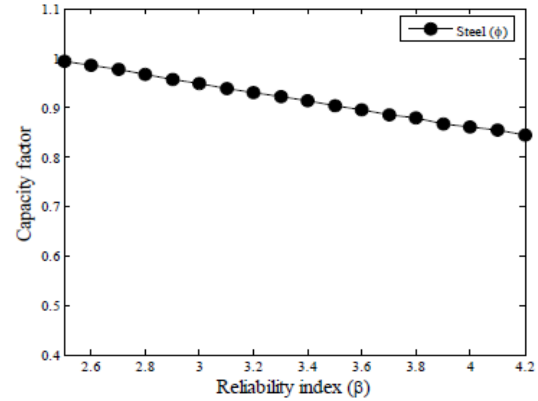


(b) ϕ*=0.95 at* β *= 3.04*

Capacity Reduction Factor

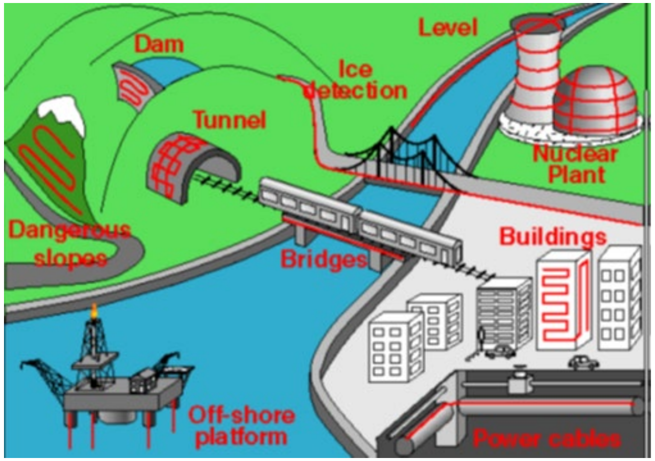


(c) ϕ*=0.94 at* β *= 3.04*



(d) ϕ*=0.94 at* β *= 3.04*

AS5100: Part 7 Rating



AS5100: Part 8 Strengthening and rehabilitation

INFORMATION ON MATERIAL CHARACTERISTICS OF OLD IRON AND STEELS

	Cast iron	Wrought steel Puddled steel	Mild steel (19^{th} century)	Mild steel (20^{th} century)
Suphur-print (Bauman n-print)	Content of sulphur is depending on coke quality	Slag, segregation containing phosphorus and sulphur	Core segregation containing phosphorus and sulphur	Low content of phosphorus and sulphur
Micro-structure ~1:400	Cast iron with lamellar graphite	Ferritic, inhomogeneous grain size distribution, oxide inclusions, slag lines	Ferritic-pearlitic, increasing grain size from the edge to the core, oxide and sulphide inclusions	Homogenous small grain
Chemical analysis	$C \approx 2.0\text{-}4.0\%$ $Mn \approx 0.2\text{-}1.2\%$ $Si \approx 0.3\text{-}3.0\%$ $S < \approx 1.2\%$ $P < \approx 1.0\%$	Very variable $C < \approx 0.08\%$ $Mn < \approx 0.4\%$ $S < \approx 0.04\%$ $P < \approx 0.6\%$	Bessemer/Thomas steel $C \approx 0.02\text{-}0.1\%$ $Mn \approx 0.3\text{-}0.5\%$ $S < \approx 0.1\%$ $P < \approx 1.0\%$ Siemens-Martin steel $C \approx 0.05\text{-}0.15\%$ $Mn \approx 0.2\text{-}0.5\%$ $S \approx 0.02\text{-}0.15\%$ $P \approx 0.03\text{-}0.06\%$	Low-allowed steel (T, SM) $C \approx 0.1\text{-}0.2\%$ $Mn \approx 0.4\text{-}0.5\%$ $Si \approx 0.01\%$
			Blast process: $N > \approx 0.01\%$ Hearth process: $N < \approx 0.01\%$	
			Bessemer steel: $Si > \approx 0.08\%$ Thomas steel: $Si < \approx 0.08\%$	
Tension test	Very brittle, almost no plasticity	No local necking	Local necking	Local necking and shear lips
Tension strength	Old cast iron $R_m \approx 90\text{-}135^1$ N/mm^2 $\varepsilon^3 \approx 0\%$	$R_e \approx 220\text{-}310$ N/mm^2 $R_m \approx 280\text{-}400$ N/mm^2 $\varepsilon \approx 5\text{-}20\%$	$R_e > \approx 220$ N/mm^2 $R_m \approx 370\text{-}400$ N/mm^2 $\varepsilon \approx 20\%$	Low-alloyed steel $R_e \approx 240\text{-}280$ N/mm^2 $R_m \approx 370\text{-}450$ N/mm^2 $\varepsilon^3 \approx 15\text{-}25\%$
Specimen after tension test	No local necking		Local necking	

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Conclusions

This lecture has set out to introduce the harmonised Australian/ New Zealand Standard AS/NZS: 5100 Part 6 on Steel and Composite Construction. In addition to highlighting the structure of the standard, the lecture highlights the introduction of high strength materials such as high strength concrete and steel. Innovations in the area of shear connector provisions are introduced and the lecture also highlights the important work carried out on assessing overseas steels and their reliability which is a major innovation in restricting barriers of trade in the area of constructional steel.

Acknowledgements

This lecture highlights the major changes to AS/NZS: 5100 Part 6. The standard was completed in 2017 and a commentary is now being developed by BD-90-06. Dr Hicks was the Chairman of this committee until Professor Uy succeeded him in 2019.

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