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AS/NZS 2327 COMPOSITE STRUCTURES: A NEW STANDARD FOR STEEL-CONCRETE COMPOSITE BUILDINGS

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ABSTRACT

This paper provides an overview of the draft joint Australian and New Zealand standard AS/NZS 2327, which will be the first harmonized standard between Australia and New Zealand for the design of steel-concrete composite buildings. Once published, this standard will replace the composite design provisions given in the existing standards AS 2327.1 and NZS 3404. As Chairs of the Committees responsible for AS/NZS 5100.6 and AS/NZS 2327, the authors of this paper present the challenges faced from the introduction of concrete compressive strengths up to 100 MPa together with quenched and tempered steels with a yield strength up to 690 MPa. Perhaps one of the most innovative aspects is that this standard is one of the few international limit state composite design standards that is underpinned by rigorous structural reliability analyses. It is anticipated that the draft for public comment document will be published in early 2016.

Introduction

This paper provides an overview of the new Australasian composite design standard for buildings AS/NZS 2327 (AS/NZS 2327 201X), which is currently being prepared for public comment and is scheduled for publication in 2016. Building on earlier steel design standard harmonization initiatives, such as the cold-formed steel structures standard AS/NZS 4600 (AS/NZS 4600 2005) and the forthcoming steel and composite bridge standard AS/NZS 5100.6 (AS/NZS 5100.6 201X), AS/NZS 2327 is the first joint Australian and New Zealand design standard for composite buildings. It has been a catalyst for further harmonization activities in design standards for steel construction and, in the future, it is hoped that this work may lead to a bringing together of the existing AS 4100 (AS 4100 1998) and NZS 3404.1 (NZS 3404.1 1997) into a joint Australian and New Zealand steel structures standard.

The project proposal submitted to Standards Australia was initially entitled "Suite of Standards for Composite Structures for use in Buildings and other non-bridge infrastructure incorporating existing AS2327.1-2003, AS2327.2-201X, AS2327.3-201X and ASS2327.4-201X". The current draft of AS/NZS 2327 consists of 10 Sections and several Appendices. The structure is presented below in the following subheadings. Where significant changes have been made compared to the existing AS 2327.1 and NZS 3404, these are highlighted and a brief overview of the background work is given.

Section 1 - Scope, Materials, Limit States And Methods Of Structural Analysis

AS/NZS 2327 is concerned with the design of steel-concrete composite members and floors in buildings. In addition, for consistency with the concrete structures design standard AS 3600 (AS 3600 2009) and NZS 3101 (NZS 3101 2006), concrete compressive cylinder strengths f'_c up to 100 MPa are permitted. Also, for consistency with the Australian steel structures standard AS 4100, structural steel with a yield strength f_y up to 690 MPa is allowed. The magnitude of these material strengths is much higher than permitted by other

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international design standards on composite construction such as Eurocode 4 (EN 1994-1-1 2004) and the North American specification for structural steel in buildings (ANSI/AISC 360-10 2010), which meant that, in the development of the appropriate design models, AS/NZS 2327 is underpinned by rigorous structural reliability analyses to enable the appropriate capacity factors to be calculated. As a consequence of this, AS/NZS 2327 is one of the few Australasian standards that responds to the National Construction Code (NCC 2015), which requires structural reliability Verification Methods to be used in demonstrating the structural reliability inherent in new documents proposed for referencing in the NCC.

A design life of 50-years is assumed. In a similar manner to North American practice, a global factor approach is adopted in Australasian standards where the design resistance is calculated by multiplying the nominal (characteristic) resistance by a capacity reduction factor ϕ (cf. the European partial factor approach, where $\phi = 1/\gamma_M$). Whilst this approach is simple to apply in structural steel and reinforced concrete design, it can prove problematical to apply in composite design as the equations for nominal capacity can consist of up to four different materials. As a consequence of this, the existing AS 5100.6 (AS 5100.6 2004) adopts a hybrid approach, where the capacity factors in composite columns are applied to the individual material components with ϕ for structural steel together with reinforcing steel and ϕ_c for concrete. This hybrid approach is maintained in AS/NZS 2327. Finally, in the interests of international harmonization, the nomenclature follows the ISO 3898 format (ISO 3898 2013), unless the change to a well-known variable could cause confusion (e.g. f'_c is maintained).

Section 2 - Design of Composite Slabs

Whilst reference was made to BS 5950-4 (BS 5950-4 1994) or Eurocode 4 in NZS 3404, to ensure that all of the required provisions to design a composite building are contained within a single document, AS/NZS 2327 provides rules for composite slabs. A wide range of sheeting types that can be used in composite slabs is permitted (see Figure 1).

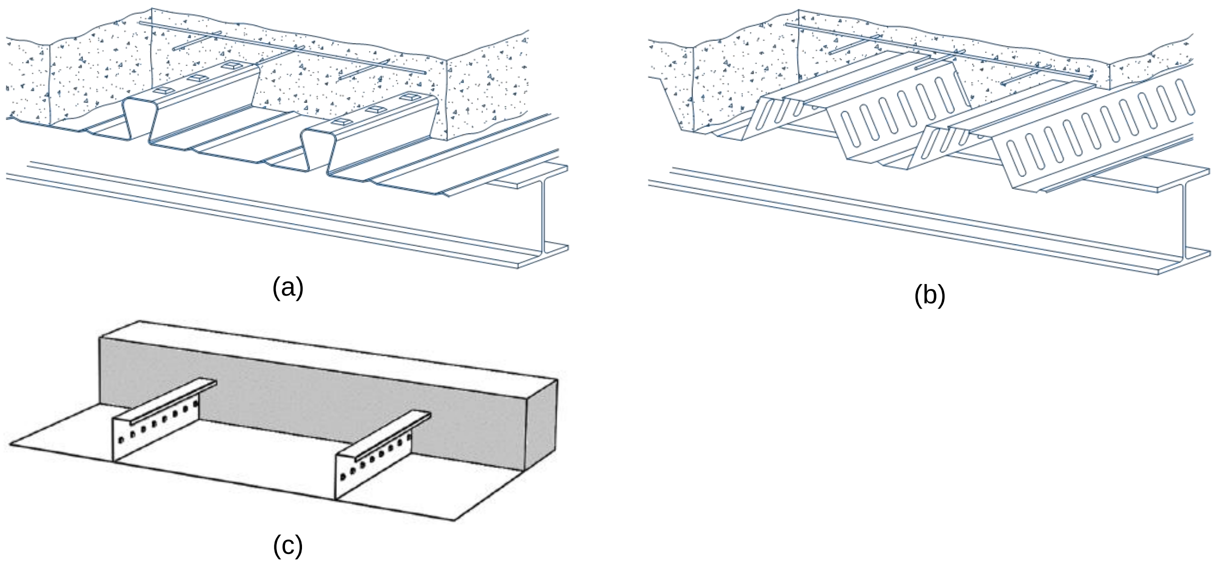

Figure 1. Profiled sheeting types permitted in AS/NZS 2327 (a) re-entrant (b) open trough (c) clipped-pan.

Whilst the traditional *m-k* method is supported, the partial connection method is promoted in AS/NZS 2327 due to the following shortcomings in the *m-k* method:

(i) The results contain all the influencing parameters, such as materials, slab geometry and composite action; however, it is not possible to separate them from one another.
(ii) The methodology is not based on a mechanical model and is therefore less flexible than the partial connection method. For example, the benefit of including reinforcement bars, end anchorage, etc. cannot be quantified unless additional tests are undertaken that include these variables.
(iii) The method of evaluation is the same whether the longitudinal shear behaviour is ductile or brittle. The use of a 0.8 penalty factor for brittle behaviour that is recommended in Eurocode 4 does not adequately reflect the advantage of using good mechanical interlock, owing to the fact that the advantage increases with span.
(iv) Other loading arrangements that differ from the test loading can be problematical.

Point (iv) is worthy of some note by designers. From investigations by the first author (Hicks 2008) it has been found that for the case when concentrated loads are applied at a distance from the support L_p that is less than that provided in the tests to evaluate the longitudinal shear resistance of the sheet L_s (i.e. $L_p < L_s$), the resistance of the composite slab can be overestimated using the *m-k* method.

Unlike Eurocode 4, the capacity factor for the longitudinal resistance of composite slabs has been directly evaluated for AS/NZS 2327. From an investigation of 61 Australasian composite slab tests, interim results from reliability analyses suggests that a capacity factor of around $\phi = 0.6$ is appropriate (which is lower than the recommended value in Eurocode 4 of $\phi = 0.8$). Furthermore, from Australian research on monitoring the long term behaviour of composite slabs it has been found that, from the presence of the profiled steel sheeting effectively acting as a 'seal' to the concrete, non-uniform shrinkage can result in higher deflections than anticipated (Al-Deen *et al.* 2015; Ranzi *et al.* 2013a; Ranzi *et al.* 2013b). As a result of this world-leading research, serviceability limit state design provisions have been developed and are presented in AS/NZS 2327.

Section 3 - Design of Composite Beams

In both Australian and New Zealand practice, historically, composite beams have been used up to simply-supported spans of 12 m. However, current and future practice will require a higher demand for ductility at the shear connection due to: longer spans; higher strength steel; fewer shear connectors (due to shape of the profiled steel sheeting and number of available positions to weld the studs); or when asymmetric steel sections are used. As a result of this, rules for the minimum degree of shear connection are provided η_{min}.

The degree of shear connection is defined by:

$$\eta = N_c / N_{c,f} \tag{1}$$

where N_c is the design value of the compressive force in the concrete given as nP_{Rd}, $N_{c,f}$ is the design value of the compressive force in the concrete with full shear connection (which is the lesser of $A_a f_{yd}$ and $0.85 f_{cd} b_{eff} h_c$), n is the number of shear connectors from the point of zero moment to the point of maximum moment, P_{Rd} is the design resistance of a shear connector ($P_{Rd} = \phi P_{Rk}$), A_a is the cross-sectional area of the steel beam, f_{yd} is the design yield strength of the steel ($f_{yd} = \phi f_y$) and f_{cd} is the design compressive strength of the concrete ($f_{cd} = \phi_c f'_c$).

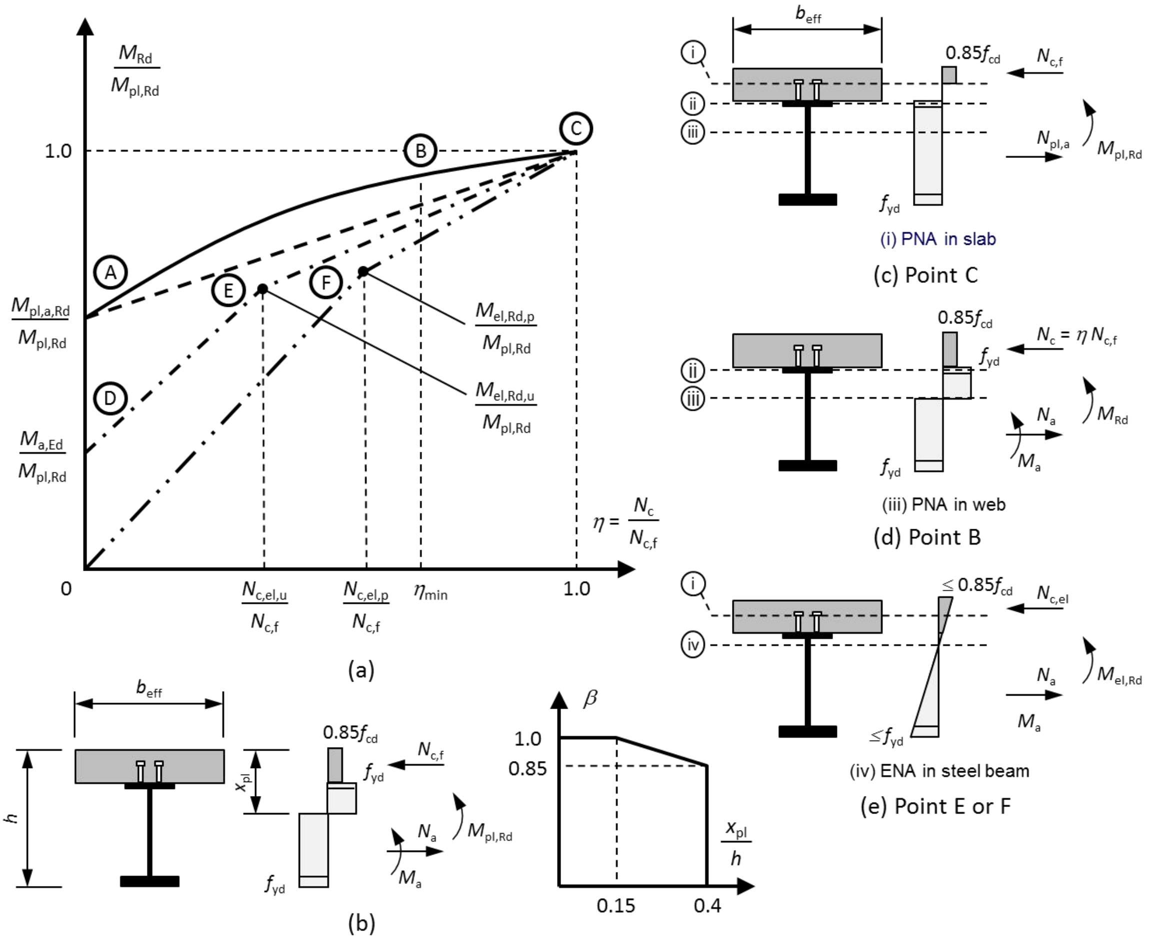

Figure 2. Design methods given in AS/NZS 2327 for simply-supported composite beams.
The bending resistance of composite beams may be evaluated using rigid plastic theory, non-linear theory and elastic analysis. The different design methods that will be permitted in AS/NZS 2327 are shown graphically in Figure 2, together with the corresponding stress distributions for a composite beam with a solid slab. Full shear connection occurs at Point C in Figure 2(a), which corresponds to η = 1.0. From equilibrium of the stress blocks, the three possible positions for the plastic neutral axis are shown by (i), (ii) and (iii) in Figure 2(c).

For cases when $\eta \geq \eta_{min}$, the simple *interpolation method* may be used, where the design moment resistance M_{Rd} is evaluated by finding η and interpolating between Point A and C in Figure 2(a) (Point A is given by the design plastic moment resistance of the structural steel section $M_{pl,a,Rd}$ alone). The *equilibrium method*, given by the convex curve ABC, is a less conservative alternative. In this case, the plastic neutral axis has two possible positions within the steel section given by (ii) and (iii) in Figure 2(d). The design lines AC and ABC are based on the assumption that the effective areas of the steel and concrete can reach their design strengths before the concrete begins to crush. AS/NZS 2327 assumes that there may be a possibility for premature crushing of the concrete if $f_y \geq 450$ MPa and the ratio x_{pl} / h is greater than 0.4. In these circumstances, the design resistance moment should be reduced by the factor β given in Figure 2(b).

In the rare cases when $\eta < \eta_{min}$, or for cases when the characteristic slip capacity of an individual shear connector is less than 6 mm, the shear connection is deemed to be ‘non-ductile’. The design line for unpropped and propped construction is given in Figure 2(a) by lines DEC and 0FC, respectively. Point F is defined by the design elastic moment resistance for propped construction $M_{el,p,Rd}$ and $\eta_{el,p}$, which corresponds to the point where the stresses in the outermost fibre of the section reach f_{cd} or f_{yd}, as shown in Figure 2(e). For Point E, initial stresses from the bending moment applied to the structural steel section $M_{a,Ed}$ during the construction stage at Point D reduce the design elastic moment resistance $M_{el,u,Rd}$ and the corresponding value of $\eta_{el,u}$.

The appropriate capacity factors for the design models presented in Figure 2 are currently under development. However, from structural steel produced by the main suppliers to the Australasian market that are independently third-party certified (to ensure that minimum levels of quality control and traceability are being maintained), interim results from reliability studies indicate that the appropriate capacity factors for steel and concrete are ϕ = 1.0 and ϕ_c = 0.75, respectively. In addition, rules for continuous composite beams are provided, which provide benefits for reducing beam depth and controlling deflections.

The most common form of shear connector in composite construction is the headed stud. A recent structural reliability study was undertaken specifically for AS/NZS 2327 and AS/NZS 5100.6 (Hicks and Jones 2013), which considered the results from 113 push tests. This work demonstrated that the following equations for the design shear capacity P_{Rd} can be used for stud connectors and bolts embedded in solid concrete slabs and encasements with $f'_c \leq 100$ MPa:

$$P_{Rd} = \phi 0.70 d_{bs}^2 f_{uc} \tag{2}$$

or

$$P_{Rd} = \phi 0.29 d_{bs}^2 \sqrt{f'_{cy} E_c} \tag{3}$$

where ϕ is the capacity reduction factor, which may be taken as ϕ = 0.8, d_{bs} is the nominal diameter of the shank of a stud connector, but 16 mm ≤ d_{bs} ≤ 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 ≤ f'_{cy} ≤ 80 MPa; E_c is the modulus of elasticity of concrete at the age being considered, which may be taken as: $E_c = \rho^{1.2}\left(0.043\sqrt{f_{cmi}}\right)$ for $f_{cmi} \leq 40$ MPa; or $E_c = \rho^{1.2}\left(0.024\sqrt{f_{cmi}} + 0.12\right)$ for $f_{cmi} > 40$ MPa, ρ is the density of concrete (kg/m³) and f_{cmi} is the mean value of the in situ compressive strength.

Section 4 - Design of Composite Columns

Although design rules for composite columns are available within AS 5100.6, many designs of Australasian buildings have historically relied on overseas standards, such as Eurocode 4. In AS/NZS 2327 the design of composite columns has been formalized and extended beyond the scope of many international standards by permitting $f'_c \leq 100$ MPa and $f_y \leq 690$MPa. This has been achieved by recalibrating the capacity factors in AS 5100.6 for a target reliability of β = 3.04 based on an extensive database of 1,583 composite column tests (Kang *et al.* 2015).

The design capacity of a composite column subjected to combined compression and bending is determined from an interaction curve (see Figure 3). The interaction curve can be obtained for a short composite column by considering several possible positions of the neutral axis within the cross-section and determining the internal forces and moments from the resulting plastic stress blocks (Figure 3(a)). However, sufficient accuracy in estimating the effects of combined compression and bending may be found by constructing the interaction curve shown in Figure 3(b) from 4 points. Under an applied force N^* equal to $\alpha_c N_{us}$, the horizontal co-ordinate $\mu_c M_s$ represents the moment due to imperfections within the column, otherwise known as the 'imperfection moment'. In this case, it is important to recognize that the moment capacity of the column has been fully utilised in the presence of the imperfection moment; the column, therefore, cannot resist any additional applied moment. However, the influence of the imperfections decreases when the axial load ratio is less than α_c, and it is assumed to vary linearly between α_n and α_c. For an axial load ratio less than α_n, the effect of imperfections is neglected. Therefore, from Figure 3, for a given applied force N^*, the available capacity to resist the applied moment is given by μM_S.

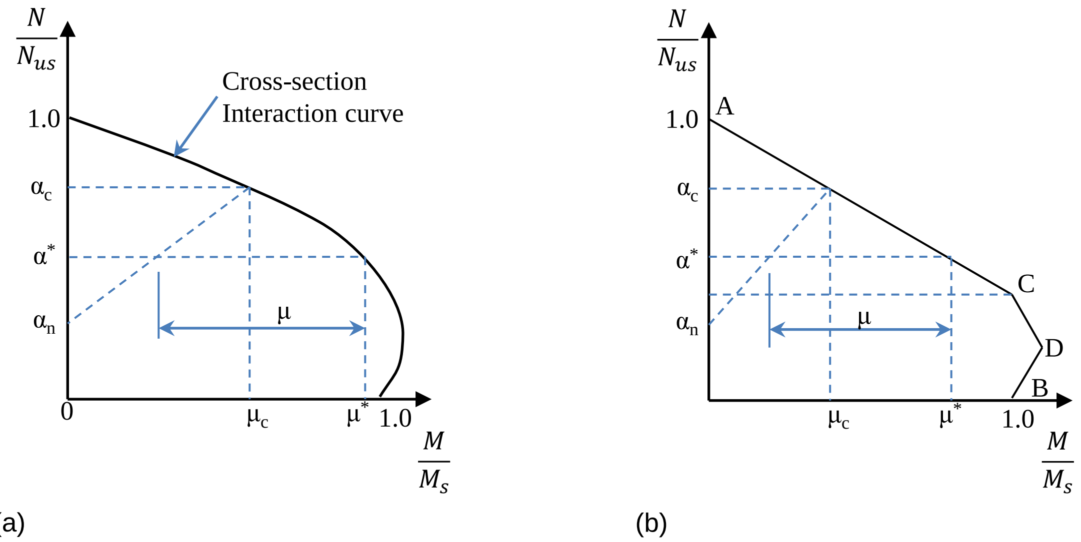

Figure 3. Compression and uniaxial bending (a) interaction curve (b) interaction curve with linear approximation.

Australasian research work on the flexural stiffness of composite columns has resulted in AS/NZS 2327 departing from other international composite design standards. From a structural reliability study based on a database of 100 composite column tests (Aslani *et al.* 2015), it is proposed to base the calculation of flexural stiffness in the new standard on the Japanese (AIJ 1997) and American reinforced concrete (ACI 318-10 2010) provisions. These rules result in lower predictions than either Eurocode 4 or AISC 360-10 and more accurately reflect the cracked section properties of composite columns.

Section 5 – Connections

Composite connections resist moment by generating a couple between their tension and compression components. The mechanics are very similar to those for non-composite moment connections, with the slab reinforcement effectively acting like an additional row of bolts in an extended end plate connection. In order to provide reliable moment-rotation characteristics, the reinforcement bars must be properly anchored and be capable of accommodating significant amounts of strain before fracture. Design rules for designing composite connections are currently being finalized within AS/NZS 2327.

Section 6 - System Design for Serviceability

Whilst significant effort is paid to limiting the deflections of the individual members, the deflection of the complete floor system can sometimes be overlooked. To remedy this situation, provisions on the system behaviour of complete floors are provided. In addition, over the last 10-years, there has been significant improvements in the development of design rules for controlling human-induced floor vibrations (Smith *et al.* 2009). In order to formalise the different procedures, design provisions are given within AS/NZS 2327.

Section 7 - System Design for Fire Resistance

Design rules on the fire design of composite slabs, composite beams and composite columns are given in the forthcoming standard. Like most of the standard, the section will offer a tiered approach from simple tabular methods to advanced methods, which may require finite element analyses. As cellular beams are becoming more popular in New Zealand these members will be covered in AS/NZS 2327; in particular, the unusual feature of the web-posts between web-openings heating up more quickly than conventional solid web beams in fire, resulting in stability considerations being required (ASFP 2010).

As well as individual member design, AS/NZS 2327 will be the first standard in the world to formalize the

Slab Panel Method (SPM) (Clifton 2006), which is otherwise known as the Cardington method (Newman *et al.* 2006) or membrane action of composite structures in fire (Vassart and Zhao 2013). The method has been specifically developed for composite floors and permits the applied fire protection to be eliminated from secondary beams, providing that the supporting primary beams are protected. The two potential yield line mechanisms are shown schematically in Figure 4.

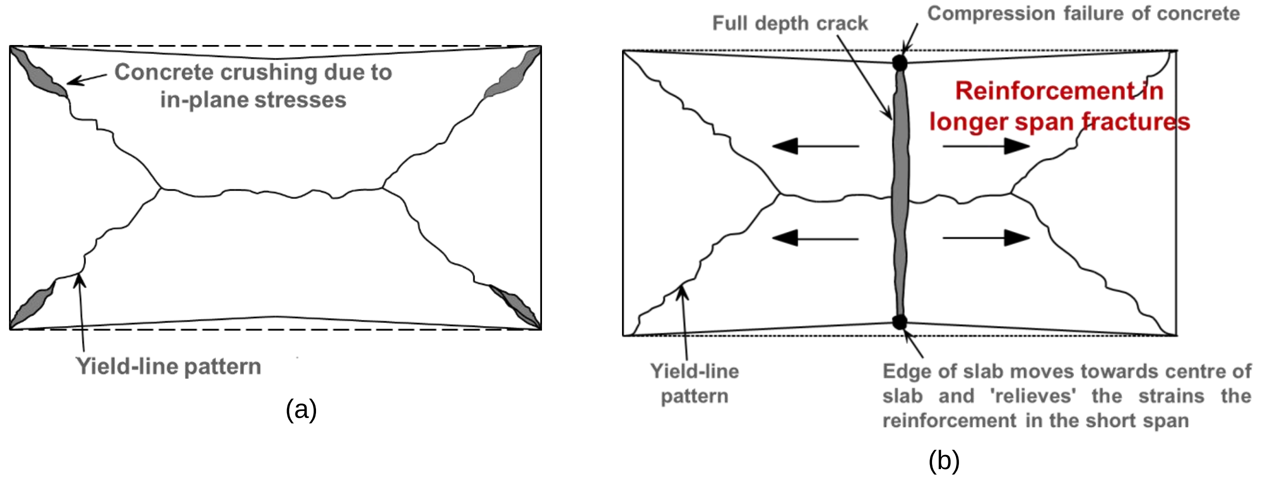

Figure 4. Slab panel yield line mechanism (a) compressive failure of concrete (b) tensile failure of reinforcement.

Section 8 – Construction

To ensure that the design assumptions remain valid (i.e. the magnitude of the capacity factors), minimum standards of workmanship are required in terms of the material and geometrical tolerances. Unlike the existing AS 4100 and NZS 3404, it is expected that instead of having a section dedicated to construction, reference will be made to the forthcoming AS/NZS 5131 (AS/NZS 5131 201X), which will reflect international best practice of specifying minimum levels of quality control and traceability of materials depending on the Importance Level of the structure (ISO/CD 17607 2015).

Section 9 - Load Testing

Due to the focus placed on design assisted by testing in composite construction, unlike AS 4100 and NZS 3404, more comprehensive provisions are given within the appendices to AS/NZS 2327.

Section 10 - System Design for Seismic Behaviour

This section has been strongly shaped by contributions from New Zealand industry, academics and practitioners. Full details on the contents of this section are presented in an accompanying paper to this conference (Cowie, 2015).

Appendices

Several appendices are given in the forthcoming AS/NZS 2327. To support long-span cellular beams, an appendix that provides design provisions for beams with regular web-openings is provided (Lawson and Hicks, 2011); as well as providing equations for designing beams of this type in ambient temperature conditions, additional thermal data can be used in conjunction with this structural model to determine limiting temperatures in fire conditions.

To facilitate future improvements to design models for composite construction, Appendices for standard tests are defined for composite slabs, profiled steel sheeting and shear connectors. Moreover, to ensure that integrity, insulation and resistance criteria are consistently achieved rules for loaded fire tests are also provided. Finally, to ensure that the required reliability indices specified in AS/NZS 1170.0 (AS/NZS 1170.0 2002), AS 5104 (AS 5104 2005) and ISO 2394 (ISO 2394 1998) for a 50-year design period are achieved, rules are provided for evaluating the design resistance of members from the standard tests described in the preceding appendices.

Conclusions

The draft AS/NZS 2327, is one of the few international limit state design standards on composite construction that is underpinned by rigorous structural reliability analyses. This was necessary, as following the international trend of using less natural resources, design rules for higher strength steel and concrete are given. The new design rules within the proposed AS/NZS 2327 provide greater alignment with international best practice and, in some cases, significant improvements are given.

Acknowledgments

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