Design of Simply-Supported Composite Beams for Strength

(To Australian Standard AS 2327.1–1996)

Design Booklet DB1.1

OneSteel Market Mills

Composite Structures Design Manual

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Disclaimer

Foreword

OneSteel is a leading manufacturer of steel long products in Australia after its spin-off from BHP Pty Ltd on the 1st November 2000. It manufactures a wide range of steel products, including structural, rail, rod, bar, wire, pipe and tube products and markets welded beams.

OneSteel is committed to providing to design engineers, technical information and design tools to assist with the use, design and specification of its products. This design booklet “Design of Simply-Supported Composite Beams for Strength” was one of the first two design booklets of the Composite Structures Design Manual, which is now being completed and maintained by OneSteel.

The initial development work required to produce the design booklets was carried out at BHP Melbourne Research Laboratories before its closure in May 1998. OneSteel Market Mills is funding the University of Western Sydney’s Centre for Construction Technology and Research in continuing the research and development work to publish this and future booklets.

The Composite Structures Design Manual refers specifically to the range of long products that are manufactured by OneSteel and plate products that continue to be manufactured by BHP. It is strongly recommended that OneSteel sections and reinforcement and BHP plate products are specified for construction when any of the design models in the design booklets are used, as the models and design formulae including product tolerances, mechanical properties and chemical composition have been validated by detailed structural testing using only OneSteel and BHP products.

To ensure that the Designer’s intent is met, it is recommended that a note to this effect be included in the design documentation.

Contents

Preface

This design booklet forms part of a suite of booklets covering the design of simply-supported and continuous composite beams, composite slabs, composite columns, steel and composite connections and related topics. The booklets are part of the OneSteel Market Mills' Composite Structures Design Manual which has been produced to foster composite steel-frame building construction in Australia to ensure cost-competitive building solutions for specifiers, builders and developers.

Simply-supported composite beams have been favoured in the construction of composite steel-frame buildings in Australia. This is essentially because simple steel connections such as the web-side-plate connection (see design booklet DB5.1 – Design of the Web-Side-Plate Steel Connection) are very economical to use when the steel frame is erected.

This design booklet contains important explanatory information and worked examples about the strength design method in Section 6 of Australian Standard AS 2327.1-1996, Composite Structures, Part 1: Simply Supported Beams. It is intended that this information will assist structural design engineers to understand the engineering principles on which the design method is based. The coverage of the strength design method is continued in design booklet DB1.2 – Design of the Shear Connection of Simply-Supported Composite Beams (To Australian Standard AS 2327.1-1996).

Design aids have already been prepared to support the use of the design method, and are included in the Composite Beam Design Handbook (in Accordance with AS 2327.1-1996) [2] published jointly by the AISC and Standards Australia. These comprise Design Tables (Appendix A) and computer software (COMPBEAM™). Although these design aids are intended to make the design process more efficient, it is essential that the users have a clear understanding of the design concepts and design rules prior to using them.

The strength design method in AS 2327.1 is based on partial shear connection strength theory and rectangular stress block theory, and is applicable to the design of composite beams with compact steel sections and ductile shear connection. Non-compact steel sections can be catered for by representing them in design as equivalent compact sections. Slender steel sections are not permitted. Details for ensuring that ductile shear connection is achieved are given in Sections 8 and 9 of AS 2327.1, and explanatory information about these rules can be found in design booklet DB1.2. Computer program COMPSHEAR™ can be used in association with COMPBEAM™ to design the shear connection in accordance with DB1.2.

The method of strength design presented for simply-supported composite beams has also been extended to cover the design of continuous composite beams, noting that very similar principles apply. The reader is referred to design booklet DB2.1 – Design of Continuous and Semi-Continuous Composite Beams with Rigid Connections for Strength, and an associated computer program COMPSECT™. Partial shear connection strength theory is also applicable to the design of composite slabs with ductile shear connection, which is also covered in a separate design booklet DB3.1 – Design of Composite Slabs for Strength. Finally, it is important to point out that the strength design method in AS 2327.1 is in harmony with leading overseas Codes, Standards and Design Specifications which address the design of composite beams.

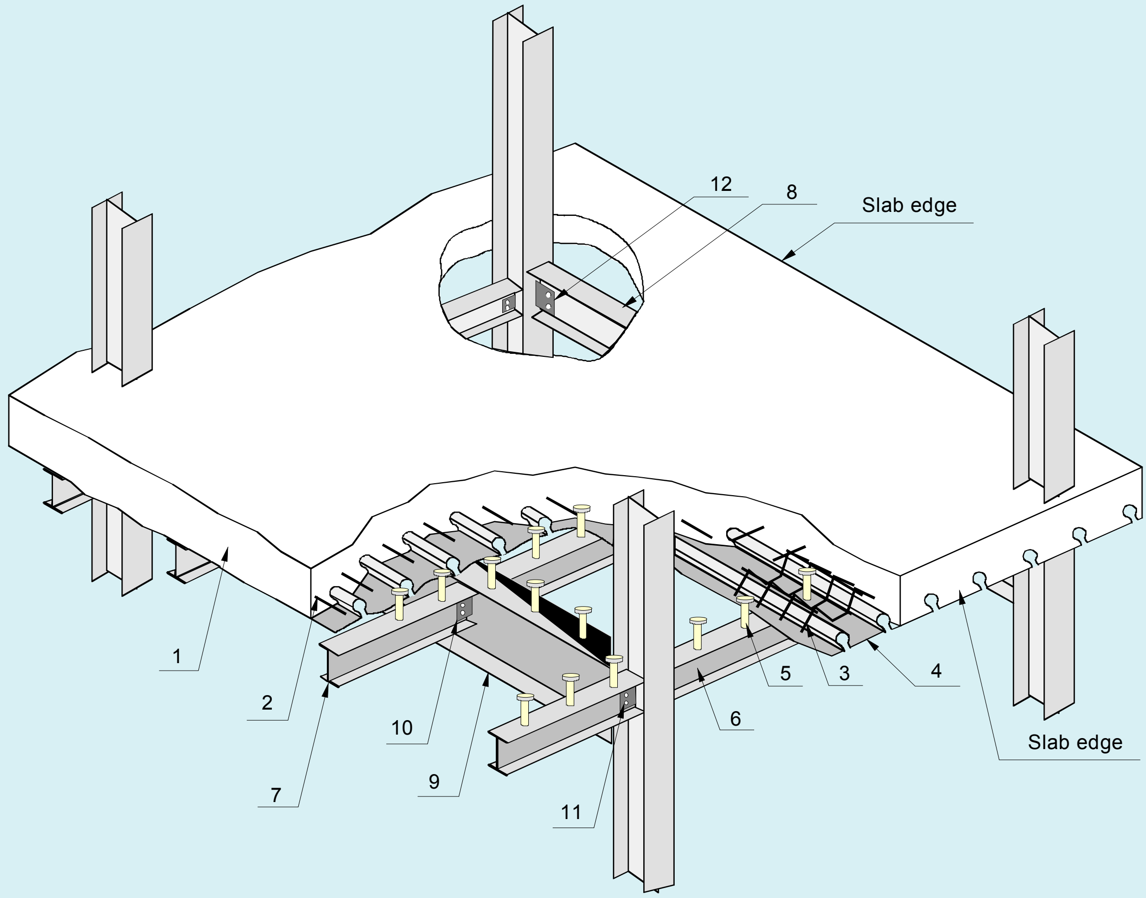
Edition 1.0 was published by BHP in May 1998. Edition 2.0 contains some minor corrections to the first edition, and is published by OneSteel.

1. SCOPE AND GENERAL

1.1 Scope

The strength design method in Section 6 of Australian Standard AS 2327.1–1996, Composite Structures, Part 1: Simply Supported Beams [1] is addressed in this design booklet.

The type of construction envisaged is shown in Fig. 1.1.



1. Slab
2. Slab reinforcement
3. DECKMESH™
4. Profiled steel sheeting
5. Shear connectors
6. Edge secondary beam
7. Internal secondary beam
8. Edge primary beam
9. Internal primary beam
10. Secondary-beam-to-primary-beam connection
11. Secondary-beam-to-column connection
12. Primary-beam-to-column connection

Figure 1.1 Members and Components of a Composite Floor

This booklet does not cover design for serviceability, design of shear connectors, design of the concrete slab for transfer of longitudinal shear or design for fire resistance. Some of these aspects will be covered in later booklets.

1.2 General

The details of the different types of components which may be used in the construction of simply-supported composite beams designed in accordance with this booklet are described in this section.

Steel Beam

The alternative types of steel beams that are permitted are shown in Fig. 1.2. The cross-section of the steel beam must be symmetrical about the vertical axis. Cold-rolled RHS, SHS and channel sections may be used provided that the wall thickness satisfies the requirements of AS 2327.1 (Clauses 5.2.3.3(a) and 8.4.3.1).

The channel sections shown in Fig. 1.2(c) and (d), and the T-sections shown in Fig. 1.2(g) and (h), may not be the most efficient steel sections for use in composite beams. However, these sections may be encountered in design when hollow sections or I-sections are notched to allow the passage of service ducts within the depth of the beams. Optional flange plates may be attached to the bottom flange of some of the steel beam types (see Fig.1.2(a)) to increase the moment capacity of the cross-section.

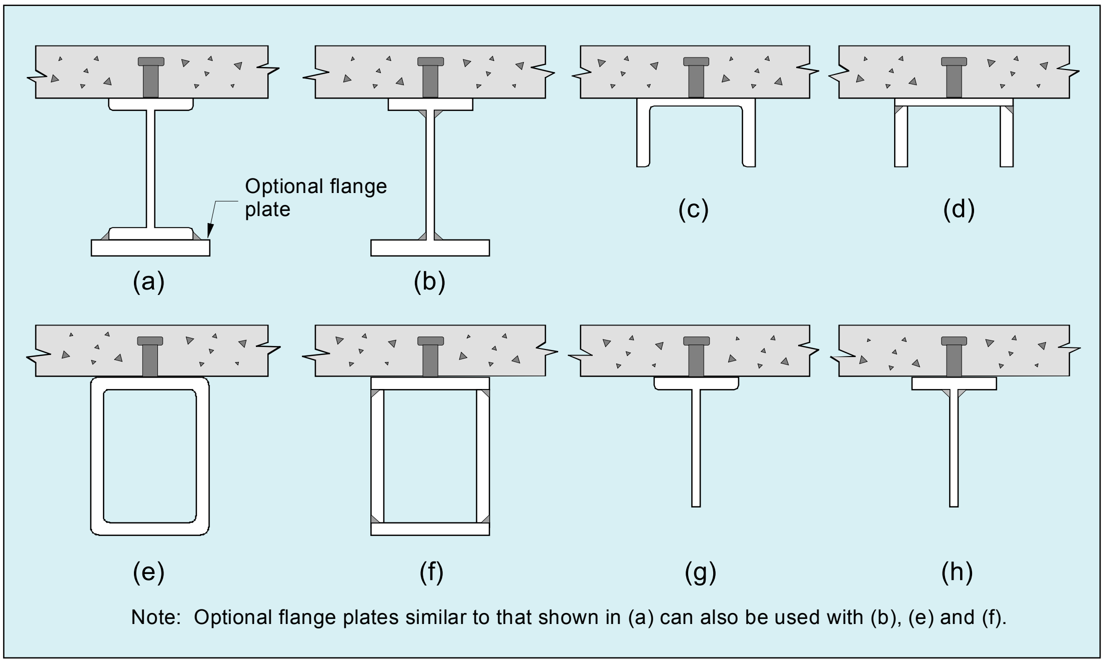


Figure 1.2 Alternative Steel Beam Types

Concrete Slab

The concrete slab forms the top flange of the composite beam. It must be reinforced with deformed bars or mesh to strengthen it against flexure, direct tension or compression, and vertical or longitudinal shear. These action effects can arise due to direct loading, shrinkage and temperature effects, fire, etc. The use of profiled steel sheeting as the bottom-face reinforcement in composite slabs can significantly reduce the amount of conventional reinforcement required in the slab for flexural or shrinkage and temperature effects. The design of solid (reinforced-concrete) slabs must be in accordance with AS 3600. Composite slabs can be designed using the information given in the design booklets provided in Part 3 of this manual. Restrictions which apply to the geometry of the profiled steel sheeting are given in Clause 1.2.4 of AS 2327.1, and, in association with other measures, were necessary to ensure that the shear connection is both efficient and ductile.

The design of composite beams with a precast concrete slab is beyond the scope of AS 2327.1 and, therefore, this booklet.

Profiled Steel Sheeting

The major types of profiled steel sheeting used in Australia, viz. BONDEK II, COMFORM and CONDECK HP (see Products Manufactured From OneSteel and BHP Steel in this manual), all satisfy the geometric requirements specified in Fig. 1.2.4 of AS 2327.1. In accordance with Fig. 1.2.4(a) of AS 2327.1, the minimum cover slab thickness ($D_c - h_r$) is 65 mm. Therefore, the minimum overall slab depth D_c of a composite slab is nominally 120 mm for BONDEK II and CONDECK HP, and 125 mm for COMFORM.

Shear Connectors

Headed studs (manually or automatically welded), channels or high-strength structural bolts shown in Fig. 1.3 may be used as shear connectors. Automatically welded headed studs are the only type of shear connector that may be attached through profiled steel sheeting.

The geometry that the shear connectors must conform with is defined in Clause 8.2.2. It should be noted that the 100TFC section is no longer produced, but the new 300PLUS, 100 PFC section may be used as a direct substitute.

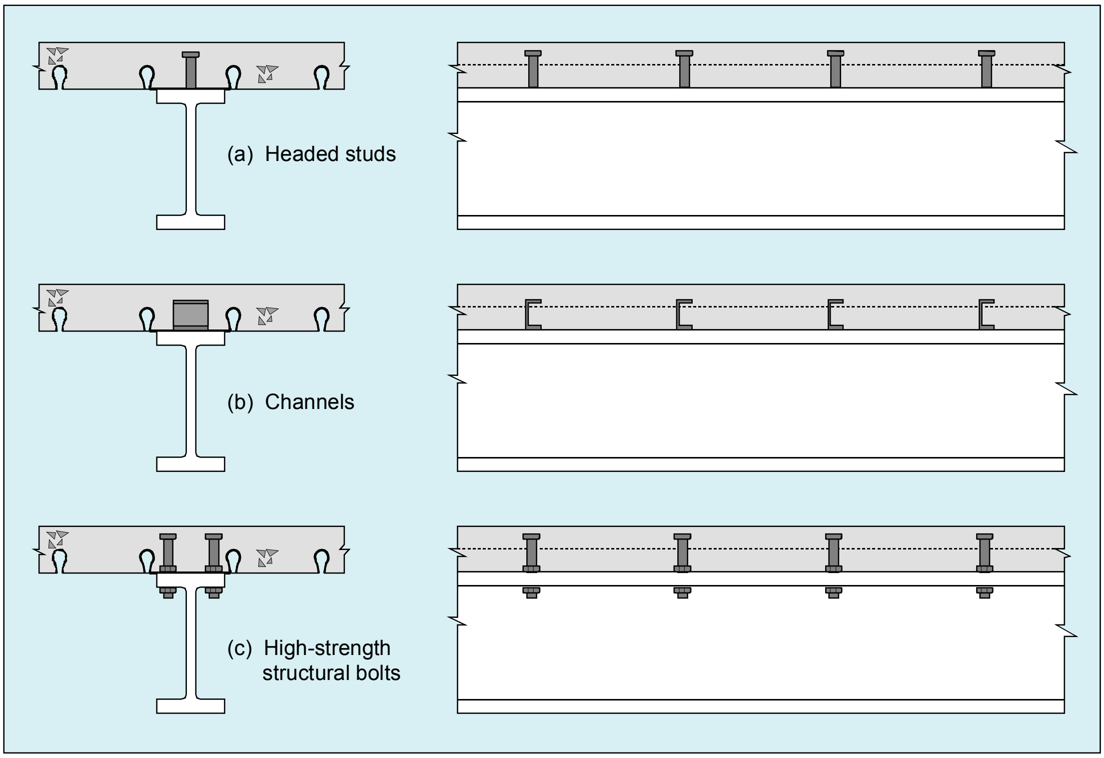


Figure 1.3 Acceptable Shear Connector Types

Steel End Connections

The most commonly used steel end connection for simply-supported composite beams is the web-side-plate connection which is shown in Fig. 1.4.

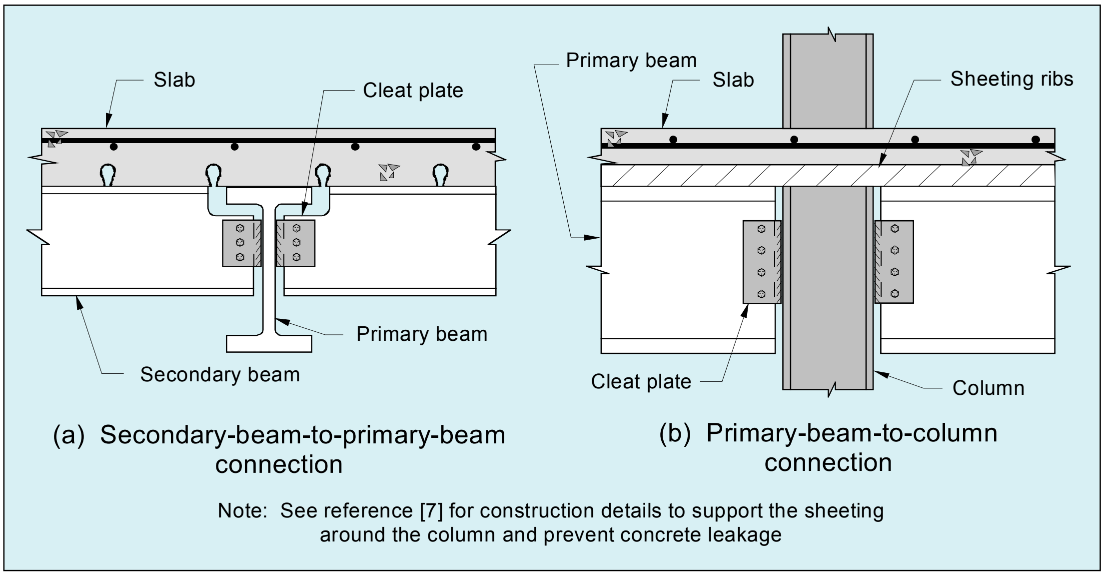


Figure 1.4 Web-Side-Plate Connection

The design of this type of steel connection is addressed in design booklet DB5.1 for the bare steel state, but not when it becomes a semi-rigid composite connection due to continuity of the slab reinforcement as shown in Fig. 1.4. In either case, it is conservative to assume simply-supported support conditions for the design of the beam. In certain types of structures, such as carparks, careful consideration should be given to controlling cracking of the concrete, and accordingly, unpropped construction may be favoured (see Clause 7.3.2 of AS 2327.1), or else the beams may be designed as continuous using design booklet DB2.1 and other types of steel connections used.

2. TERMINOLOGY

Some important terminology used in this booklet is summarised in this section. Reference should also be made to Clause 1.4.3 of AS 2327.1 for additional terminology.

Complete Shear Connection (β = 1)

The condition where the moment capacity of the cross-section of the composite beam is not governed by the strength of the shear connection.

Composite Action

Interaction between the steel beam and the concrete slab to resist action effects as a single structural member; assumed to commence when the concrete in the slab has attained a compressive strength of at least 15 MPa. It is assumed to be fully developed once the compressive strength of the concrete attains its specified design value f'_c.

Composite Slab

A cast-in-situ concrete slab which incorporates profiled steel sheeting as permanent soffit formwork.

Construction Stages

The following Construction Stages are identified in AS 2327.1 (see Clause 4.2 and Appendix F):

Stage 1: While the steelwork is being erected; falsework and/or props are possibly installed; and profiled steel sheeting is placed and attached to the steel beams. Construction loads are carried by the steel beams acting alone, the strength of which should be determined giving due consideration to the lateral restraint provided during this stage. Generally, this is a critical stage in the design of the steel beams.

Stage 2: Attachment of shear connectors, fixture of reinforcement, and possibly installation of props (if not installed during Stage 1) is undertaken in readiness for concreting. As for Stage 1, the loads are carried by the steel beams acting alone which may or may not be propped.

Stage 3: Period between commencement of concreting and initial set of the concrete. Composite action is not available. In unpropped construction, all the construction loads corresponding to Stage 3, including the wet concrete load, are carried by the steel beams acting alone, and hence this should be considered as a critical stage in the design of the steel beams.

Stage 4: Period after initial set of the concrete until its compressive strength f'_{cj} reaches 15 MPa, which corresponds to the development of composite action. No additional loads should be placed on the concrete to ensure that the shear connection is not damaged during this sensitive period, which may require back-propping of beams and/or slabs.

Stage 5: Period until the concrete compressive strength f'_{cj} reaches f'_c (i.e. $15 \leq f'_{cj} < f'_c$). Removal of slab formwork/falsework or props to the steel beams or slabs may occur during this stage. With composite action initially developed, the strength of the beam may be assessed using appropriate values for the compressive strength of the concrete (see Clause 6.4.2 of AS 2327.1) and the design shear capacity of the shear connectors.

Stage 6: The remaining period of construction until the structure goes into service. The design strength of the composite beams has been reached. The in-service loads are yet to be applied, but appropriate construction loads should be considered.

In-Service Stage: The structure is occupied and carrying the in-service loads.

Minimum nominal construction loads for each of the construction stages defined above are specified in Clause 4.1.1 and Paragraph F2 of AS 2327.1.

Degree of Shear Connection (β)

The value obtained when the compressive force in the concrete at the strength limit state (F_{cp}) is divided by the compressive force in the concrete corresponding to complete shear connection in the absence of vertical shear force (F_{cc}), i.e. $\beta = F_{cp} / F_{cp}$. While F_{cc} is a property of the composite section, F_{cp} depends on the strength of the shear connection between the cross-section being designed and the ends of the beam.

End of Composite Beam

Normally this would mean the physical end of the steel beam. This is obviously the case where a steel beam end is supported by a simple steel connection such as that shown in Fig. 1.4. However, if the steel beam cantilevers past its support ,such as could occur if the steel beam rests on another steel beam or a masonry wall, then in this case the end of the composite beam only coincides with the end of the steel beam if the cantilever is not subjected to significant negative bending.

Plastic Neutral Axis

The location in the steel beam at the top of the tensile stress block assuming rectangular stress blocks.

Potentially Critical Cross-section

A transverse cross-section that is likely to be critical, the strength of which must be checked during design. Cross-sections may be critical in bending, shear or combination of both..

Shear Ratio (γ)

The ratio at a cross-section of the design vertical shear force (V^*) to the design vertical shear capacity (ϕV_u).

Solid Slab

A concrete slab with a flat soffit and without a haunch, cast in-situ on removable formwork and reinforced in accordance with AS 3600.

Tributary Area

The plan area of formwork or slab from which dead and live loads acting on the formwork or slab shall be assumed to be received by a supporting composite beam, determined in accordance with Paragraph H2 of AS 2327.1.

3. DESIGN CONCEPTS

3.1 Shear Connection

General

The interconnection between the steel beam and concrete slab of a composite beam, which enables the two components to act together as a single structural member, is broadly referred to as the "shear connection". It comprises the shear connectors, slab concrete and longitudinal shear reinforcement. Design of the shear connection is dealt with in detail in design booklet DB1.2.

The shear connection mainly resists longitudinal shear, while tensile forces can also develop in the shear connectors, particularly in the vicinity of web penetrations due to Vierendeel action or where secondary beams apply "hanging" loads to primary beams.

The shear connection resists interface slip (see Fig. 3.1) and causes the concrete slab and steel beam to interact. As a result, resultant compressive and tensile forces develop in the slab and beam, respectively.

Depending on the strength and stiffness of the shear connection, a composite beam can be significantly stronger and stiffer in flexure than the corresponding non-composite beam. Typically, the design moment capacity and second moment of area of a composite beam with complete shear connection can be 2-3 times higher than those of the steel beam alone (see Appendix A of Composite Beam design Handbook [2]).

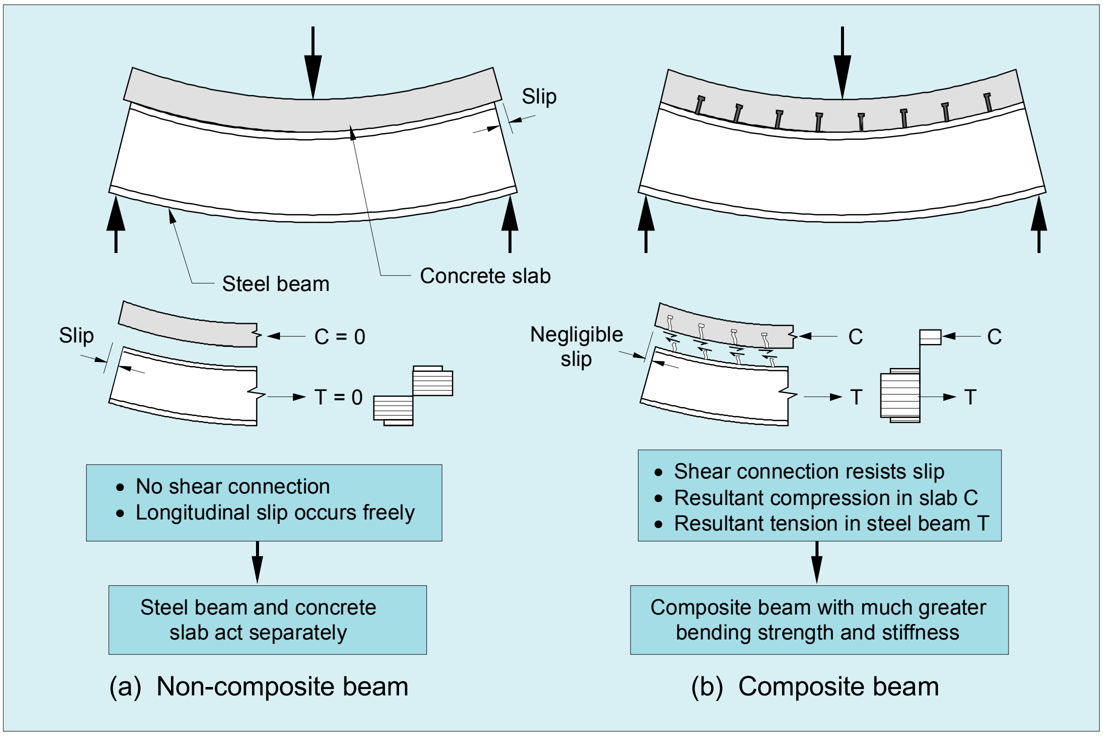


Figure 3.1 Shear Connection Resisting Longitudinal Shear

Ductile Behaviour

The strength design method given in AS 2327.1 requires that the shear connection is ductile. Generally, individual shear connectors should have a slip capacity of at least 6 mm for the behaviour of the shear connection to be considered as ductile [3]. Ductile and brittle behaviour of shear connectors, as well as the assumed model for design, are shown diagrammatically in Fig. 3.2.

Factors that affect the ductility of the shear connection include the following:

(a) type of shear connectors;

(b) type of slab, i.e. solid or composite;

(c) type and orientation of profiled steel sheeting;

(d) detailing of shear connectors, e.g. number of connectors per pan, proximity to profiled steel sheeting ribs, etc.;

(e) concrete and shear connector material properties; and

(f) detailing of longitudinal shear reinforcement.

The requirements specified in AS 2327.1 in relation to the above factors ensure that a ductile shear connection is achieved.

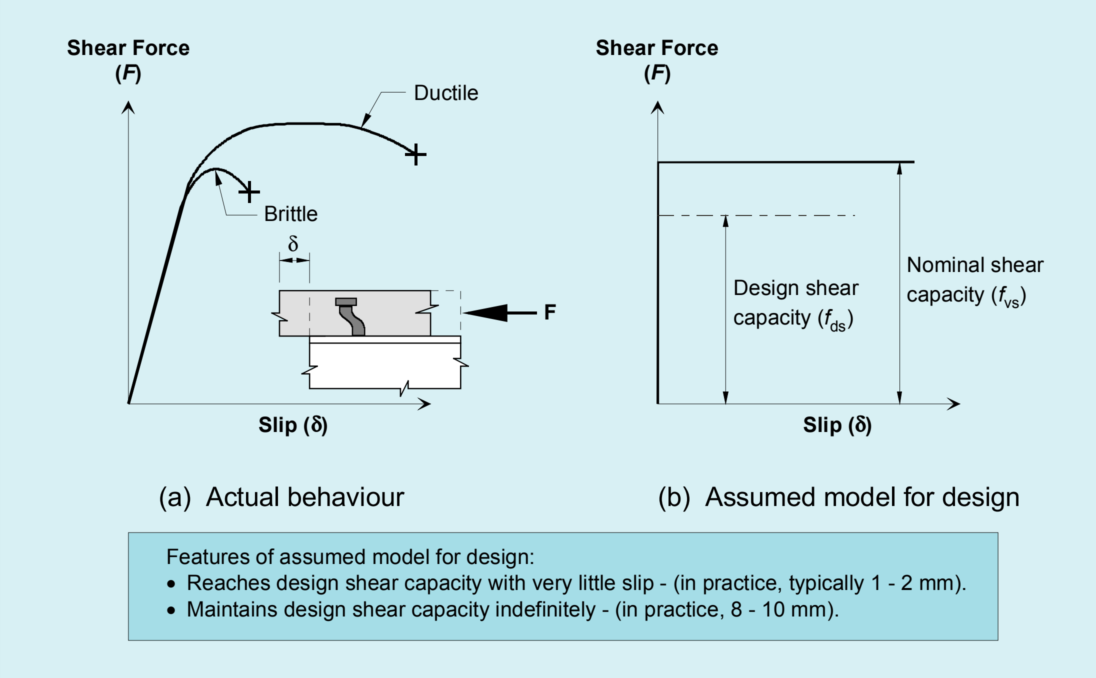


Figure 3.2 Behaviour of Shear Connectors

3.2 Beam Bending

Beam End Segment

In strength design, the design action effects M^* and V^* at a cross-section are calculated by considering equilibrium of the beam end segment taken between the cross-section of concern, and the adjacent beam end as shown in Fig. 3.3(a). Equilibrium of internal forces at the cross-section, and equilibrium of the slab included in the beam end segment, as shown in Fig. 3.3(b), are considered in the calculation of the nominal moment capacity of the cross-section M_b .

Effective Section

The moment capacity of a composite beam cross-section is calculated using the concept of an effective section, which includes an effective width of concrete flange and an effective portion of steel beam. The presence of compressive reinforcement in the slab is normally ignored in the calculation. The capacity of the profiled steel sheeting to carry force is also neglected.

The effective width of the concrete flange is affected by the in-plane shear flexibility of the concrete slab, and therefore mainly depends on the:

(a) effective span (L_{ef});

(b) centre-to-centre spacing of adjacent beams (b_1 , b_2); and

(c) overall thickness of the slab (D_c).

In the case of a solid slab, the entire cross-sectional area of the slab within the effective width is available to form part of the effective section. However, for a composite slab, the portion of the slab included depends on the orientation of the sheeting ribs. A composite slab with profiled steel sheeting that complies with the requirements of AS 2327.1(Clause 1.2.4) can be treated as a solid slab when the sheeting ribs are deemed parallel to the longitudinal axis of the beam.

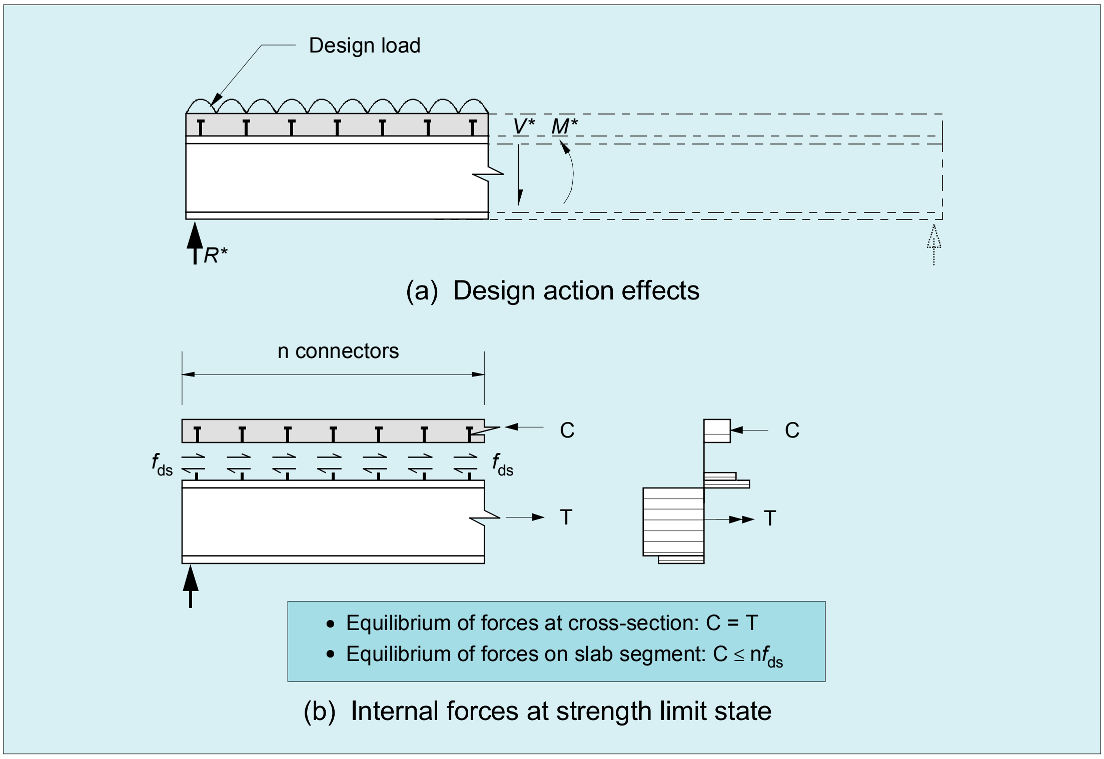


Figure 3.3 Beam End Segment

The effective portion of the steel beam is determined taking into account the effects of local buckling. Plate elements of the steel beam that are fully in tension are considered to be fully effective. In addition, the effective portion of each plate element either fully or partially in compression is included in the effective section.

The effective portion of each plate element in compression is determined on the basis of the value of its plate element slenderness (λ_e) in relation to the plate element plasticity and yield slenderness limits (λ_{ep} and λ_{ey}), which are given in Table 5.1 of AS 2327.1 and are similar to those given in AS 4100. Compact plate elements ($\lambda_e \leq \lambda_{ep}$) are considered fully effective. For non-compact plate elements ($\lambda_{ep} < \lambda_e \leq \lambda_{ey}$), only the portion that can be considered compact is included. Steel beam sections with slender plate elements ($\lambda_e > \lambda_{ey}$) are not covered. Reference could be made to Eurocode 4 [3] or BS 5950: Part 3 [4] for the design of composite beams with slender steel beam sections.

When calculating the effective portion of the steel beam, the depth of the compressive zone in the steel beam must be known. However, this is affected by the degree of shear connection (β) which varies along the length of the beam. Initially, the extent of the compressive zone can be determined based on the conservative assumption that only the steel beam is present and there is no composite action (see Fig. E2 of AS 2327.1). Although the calculation can be refined iteratively when the degree of shear connection at each potentially critical cross-section for bending is known, this is not normally done.

Rectangular Stress Block Theory

The nominal moment capacity of a composite beam cross-section (M_b) is calculated using its effective section and rectangular stress block theory.

The rectangular stress block theory is based on the following assumptions and calculation principles (see Paragraph D2.3.1 of AS 2327.1):

(a) The effective section is calculated in accordance with Clause. 5.2 of AS 2327.1.

(b) The tensile strength of the concrete is zero.

(c) A uniform compressive stress of $0.85f_c^{'}$ is developed in the concrete flange starting from the top surface of the slab.

(d) The compressive force in the concrete flange cannot exceed the longitudinal shear force which can be transferred by the shear connection between the steel beam and the concrete slab at the strength limit state.

(e) The part of the steel beam in tension is stressed to f_{yf} or f_{yw} , as appropriate.

(f) The part of the steel beam in compression is stressed to f_{yf} or f_{yw} , as appropriate.

(g) The resultant tensile and compressive forces at the cross-section are in equilibrium with each other, and therefore, the force in the concrete cannot exceed the nominal tensile capacity of the steel beam.

The stress distributions determined using rectangular stress block theory are shown in Fig. 3.4 for two typical cases. The case in Fig. 3.4(a) corresponds to complete shear connection. Complete shear connection can exist even if there is interface slip. The plastic neutral axis lies at the top face of the steel beam. The case in Fig. 3.4(b) corresponds to either complete or partial shear connection with the plastic neutral axis located within the depth of the steel beam.

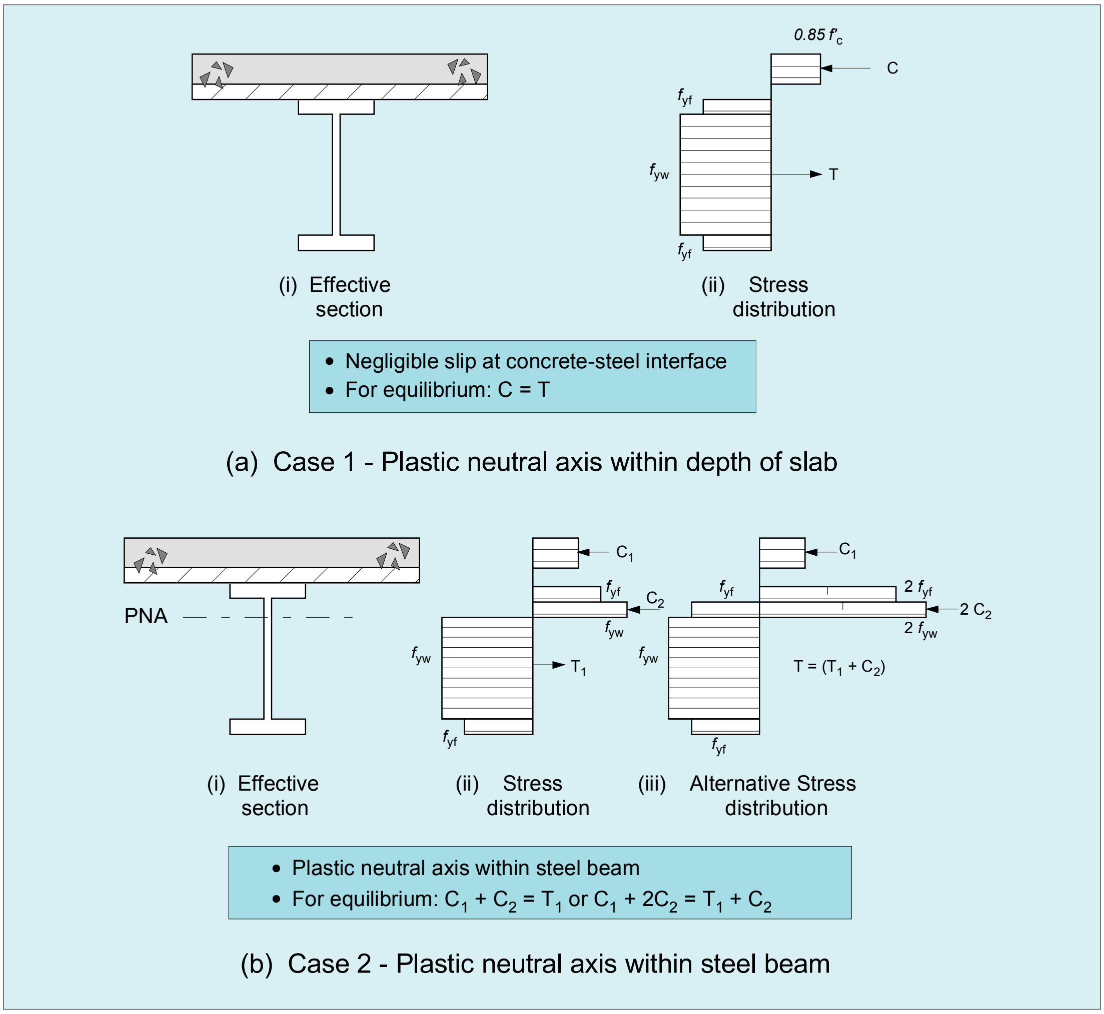


Figure 3.4 Typical Stress Distributions for Nominal Moment Capacity Calculation

Nominal Moment Capacity at a Cross-Section

A typical curve relating nominal moment capacity (M_b) at an internal composite beam cross-section with the number of shear connectors (n) between the cross-section and the adjacent end of the beam is shown in Fig. 3.5(b).

If the slab is not attached to the steel beam with shear connectors, then there is no composite action, and, ignoring any contribution from the slab, the nominal moment capacity of the beam at any cross-section equals that of the steel beam section (M_s) (see Point A in Fig 3.5(b)).

If the number of shear connectors (n) on the beam end segment shown in Fig. 3.5(a) is gradually increased, the nominal moment capacity at the cross-section x-x increases until M_{bc} is reached when $n = n_c$ (see Point B in Fig. 3.5(b)). At this point, the maximum nominal moment capacity that can be derived from the cross-section has been reached, and it is governed either by yielding of the steel beam or crushing of the concrete flange. Therefore, the nominal moment capacity at the cross-section x-x cannot be increased further by increasing the number of shear connectors on the beam end segment.

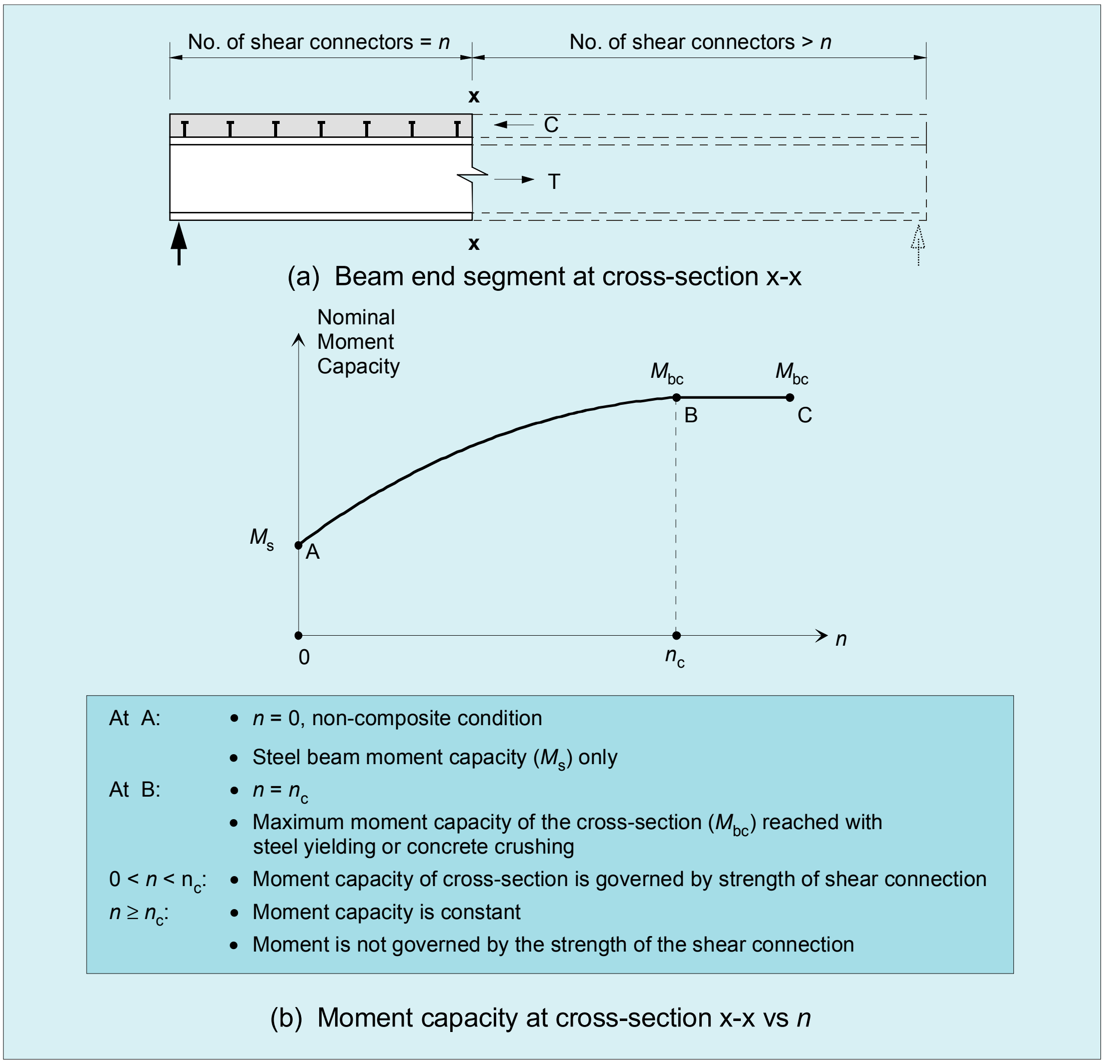


Figure 3.5 Nominal Moment Capacity at a Cross-Section

Complete and Partial Shear Connection

A composite beam cross-section is considered to have complete shear connection when the nominal moment capacity of the cross-section (M_{bc}) is not governed by the strength of the shear connection. This condition is achieved when $n \geq n_c$, as shown in Fig. 3.5(b).

Similarly, a composite beam cross-section is considered to have partial shear connection when the nominal moment capacity (M_b) is governed by the strength of the shear connection. The cross-section x-x shown in Fig. 3.5(a) has partial shear connection when $0 < n < n_c$.

It should be noted that the terms "complete shear connection" and "partial shear connection" are strictly used in relation to a cross-section, not to the beam as a whole.

Loosely, a beam can be said to have complete shear connection if one or more of its cross-sections has complete shear connection. Similarly, a beam can be said to have partial shear connection if none of its cross-sections have complete shear connection. Several cases of beams with complete and partial shear connection are shown in Fig. 3.6.

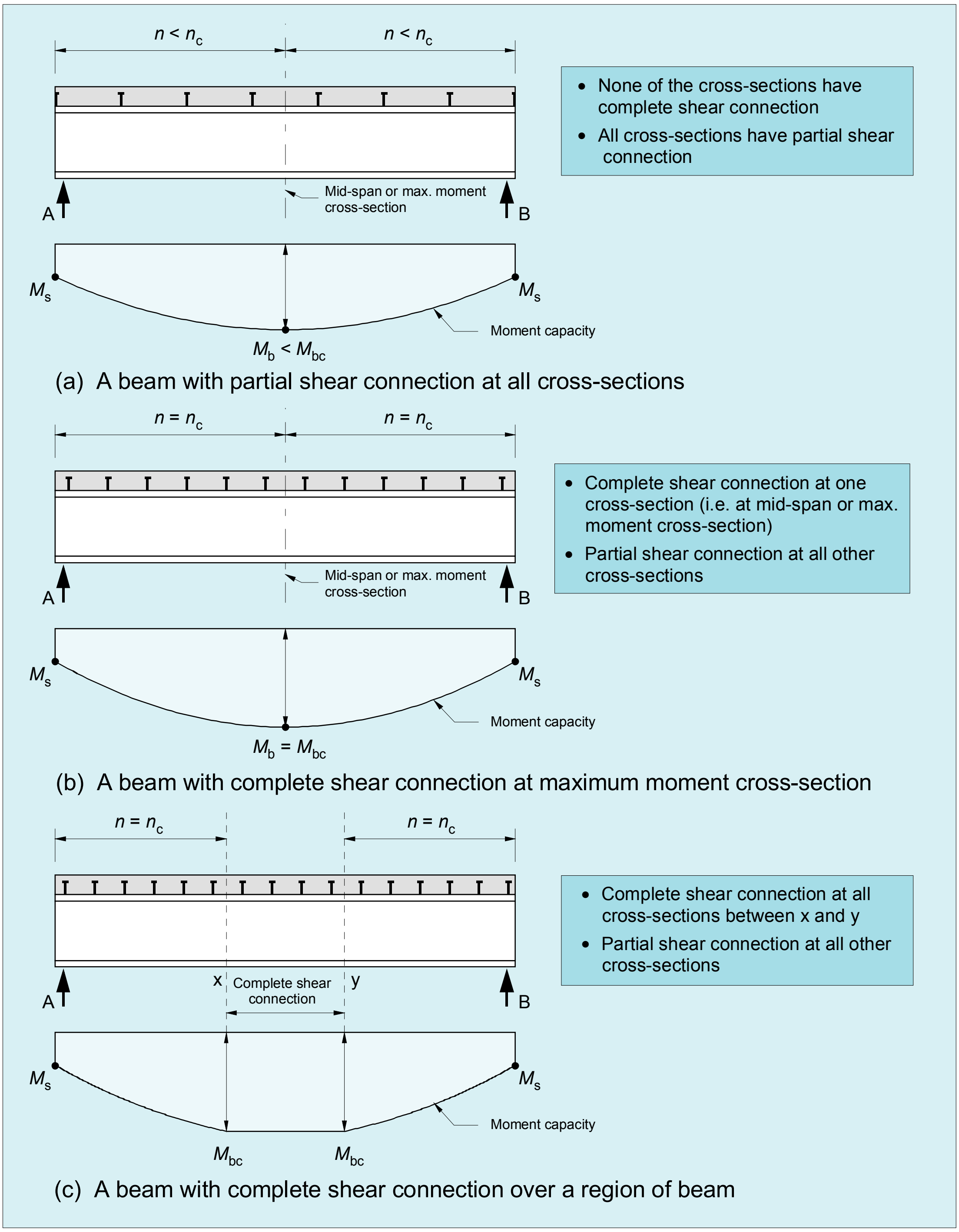


Figure 3.6 Beams with Complete and Partial Shear Connection

Degree of Shear Connection (β)

The degree of shear connection (β) is a non-dimensional parameter used to quantify the strength of the shear connection at a composite beam cross-section. It is defined using the compressive force in the concrete flange. When there is complete shear connection $\beta = 1.0$, while $\beta = 0$ when there is no shear connection. For partial shear connection $0 < \beta < 1.0$.

The degree of shear connection (β) at a cross-section is defined as:

$$\beta = \frac{F_{cp}}{F_{cc}}, \quad 0 \le \beta \le 1.0 \tag{3.1}$$

where -

F_{cp} = compressive force in the concrete flange at a cross-section with partial shear connection ($F_{cp} = nf_{ds}$, but not greater than F_{cc}); and

F_{cc} = compressive force in the concrete flange at a cross-section corresponding to complete shear connection.

The force F_{cc} is independent of the strength of the shear connection. It can be calculated assuming complete shear connection as shown in Fig. 3.4(a).

The vertical and horizontal axes of the nominal moment capacity (M_b) vs number of shear connectors (n) curve shown in Fig. 3.5 can now be converted to design moment capacity (ϕM_b) and degree of shear connection (β), and the general $\phi M_b - \beta$ curve represented as shown in Fig. 3.7. This curve can be used to determine the minimum degree of shear connection and corresponding minimum number of shear connectors required for a given design moment (M^*).

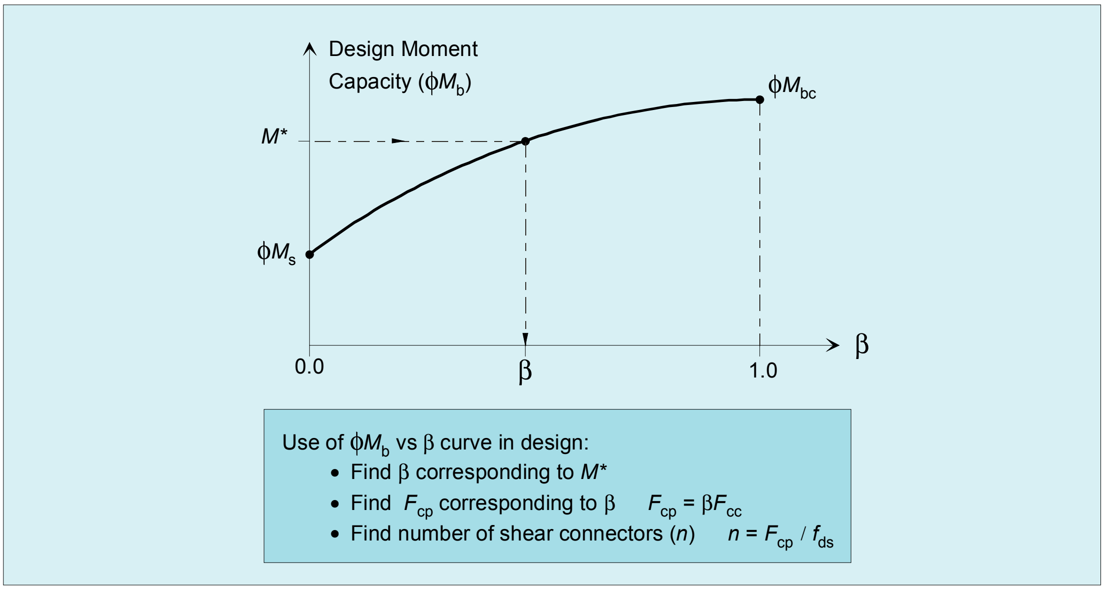


Figure 3.7 Design Moment Capacity (ϕM_b) as a Function of β

3.3 Design Vertical Shear Capacity (ϕV_u)

The vertical shear in a composite beam is mainly carried by the steel beam. A notable exception to this rule, however, is in the region of large web penetrations, the design of which will be covered in a later booklet. Unless it can be demonstrated that the contribution from the concrete slab is significant (such as in beams with large web penetrations), vertical shear is assumed to be carried by the web(s) of the steel beam (see Clause 6.4.1 of AS 2327.1).

If the contribution from the slab is ignored, then the design shear capacity (ϕV_u) of a composite beam cross-section is calculated using Clause. 5.11 of AS 4100.

3.4 Moment-Shear Interaction

At a composite beam cross-section, the web(s) of the steel beam will normally be subject to combined vertical shear and bending. The design moment capacity of such a cross-section is assumed to be reduced once the shear stresses in the steel web(s) are sufficiently large. Shear reduces the capacity of the web(s) to support longitudinal stresses induced by bending. This effect is termed moment-shear interaction. The general term used for the nominal moment capacity of a composite beam cross-section exhibiting moment-shear interaction is M_{bv}. The term M_b is used when shear does not influence the value of the nominal moment capacity.

The magnitude of the design vertical shear force (V^*) measured in relation to the design shear capacity (ϕV_u) at a cross-section is quantified using the shear ratio (γ), which is defined as:

$$\gamma = \frac{V^*}{\phi V_u}, \quad 0 \le \gamma \le 1.0 \qquad (3.2)$$

where -

V^* = design vertical shear force; and

ϕV_u = design vertical shear capacity.

It is assumed that the design moment capacity of a cross-section is not affected by vertical shear if $0 \le \gamma \le 0.5$, and is represented by the general term ϕM_b. When $\gamma = 1.0$, the design moment capacity (ϕM_{bf}) is calculated only taking account of the steel beam flanges and the concrete slab. The web of the steel beam is ignored since they are assumed to be fully utilised carrying vertical shear. The design moment capacity (ϕM_{bv}) for cases where $0.5 < \gamma < 1.0$ is calculated by linear interpolation between the points corresponding to γ values of 0.5 and 1.0.

It follows from the discussion above that in the most general case, design moment capacity (ϕM_{bv}) is a function of not only degree of shear connection (β) as discussed in Section 3.2, but also shear ratio (γ).

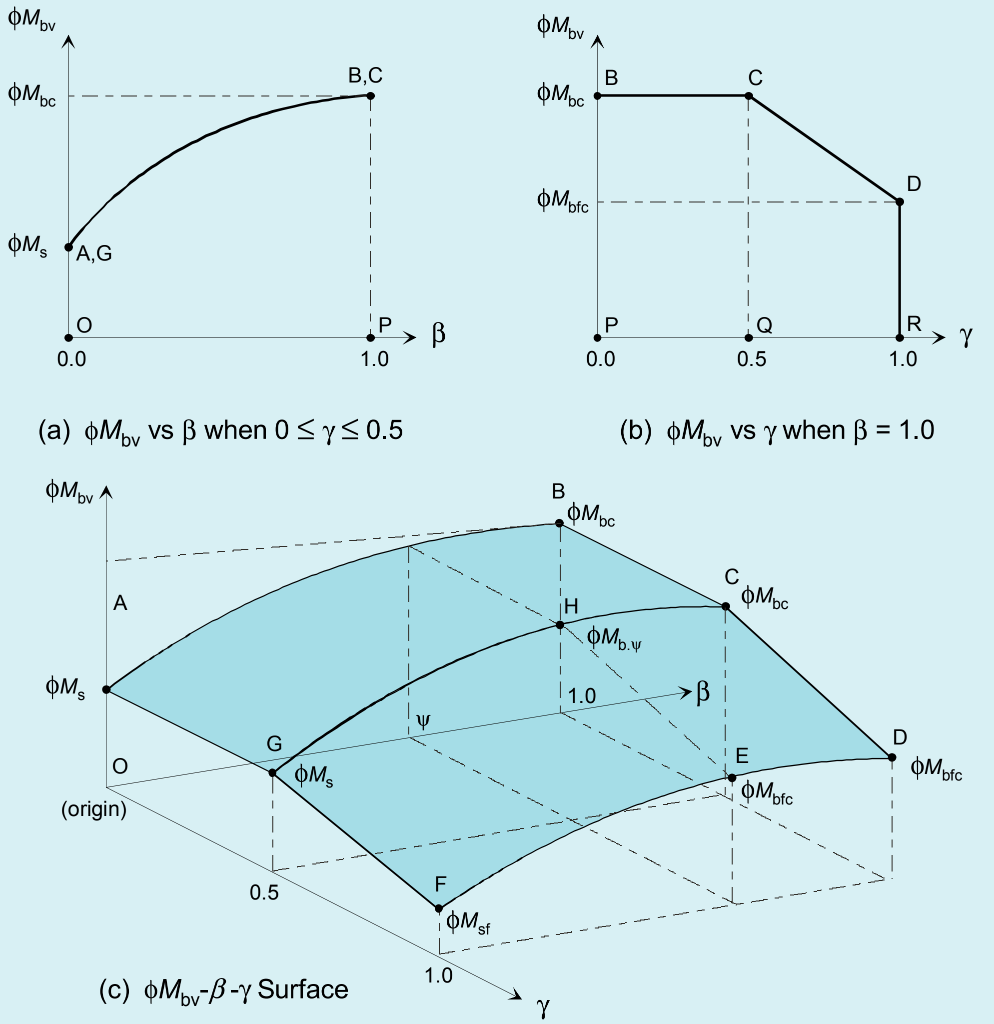
3.5 Design Moment Capacity (ϕM_{bv}) as a Function of β and γ

The three-dimensional relationship between design moment capacity (ϕM_{bv}), degree of shear connection (β) and shear ratio (γ) is explained (see Fig. 3.8).

A typical ϕM_b vs β curve when $0 \le \gamma \le 1.0$ is shown in Fig. 3.8(a). When the degree of shear connection is zero (i.e. $\beta = 0$), only the design moment capacity of the steel section (ϕM_s) is available. The maximum design moment capacity (ϕM_{bc}) is obtained when $\beta = 1.0$.

The moment-shear interaction relationship (see Section 3.4) corresponding to $\beta = 1.0$ is represented by curve BCDR in Fig. 3.8(b). The design moment capacity (ϕM_{bv}) is only reduced due to shear when $\gamma > 0.5$. When $\gamma = 1.0$ (and $\beta = 1.0$), the design moment capacity (ϕM_{bfc}) is calculated assuming that the web can only support shear, and is not available for bending.

The complete ϕM_b-β-γ surface, including the notation used for major values of design moment capacity (ϕM_{bv}), is shown in Fig. 3.8(c). Bi-linear approximations to the concave upward curves AB and GC shown in Fig. 3.8 are permitted in AS 2327.1 to simplify design calculations. When design involves cases with $\gamma \le 0.5$, curves AB and GC are approximated by a straight line between ϕM_s and $\phi M_{b.5}$, and another from $\phi M_{b.5}$ to ϕM_{bc} (see Fig. D2.2 in AS 2327.1). Similarly, when design involves cases with $\gamma > 0.5$, curves GHC and FE are approximated by straight lines between ϕM_s and $\phi M_{b.\psi}$ (GH), $\phi M_{b.\psi}$ and ϕM_{bc} (HC), and ϕM_{sf} and ϕM_{bfc} (FE) (see Fig. D3.3 in AS 2327.1).



(a) ϕM_{bv} vs β when $0 \leq \gamma \leq 0.5$

(b) ϕM_{bv} vs γ when β = 1.0

(c) ϕM_{bv}-β-γ Surface

Position on Surface	Design Moment Capacity	β	γ	Cross-section used
A G	ϕM_s	0.0 0.0	0.0 0.5	Steel
F	ϕM_{sf}	0.0	1.0	Steel without web
B C	ϕM_{bc}	1.0 1.0	0.0 0.5	Composite
D E	ϕM_{bfc}	1.0 ψ	1.0 1.0	Composite without web
H	$\phi M_{b.\psi}$	ψ	0.5	Composite

Figure 3.8 Design Moment Capacity (ϕM_{bv}) as a Function of β and γ

3.6 Effect of Propping on Design Moment Capacity

The design moment capacity of the cross-sections of a composite beam with compact cross-sections and ductile shear connection is assumed to be independent of the sequence and method (i.e. propped or unpropped) of construction. Therefore, this is not a consideration when designing for strength in accordance with Section 6 of AS 2327.1.

In unpropped construction, the steel beam supports the dead load (i.e. self-weight of the steel beam and slab) and construction live load during Construction Stages 3 and 4. Therefore, the steel beam is stressed and has deflected before composite action begins from Construction Stage 5 onwards. However, these initial locked-in stresses in the steel beam do not affect the moment capacity of the critical cross-section of the composite beam, since yielding of the steel beam at the strength limit state causes a redistribution of stresses to occur internally.

In propped construction, either the slab or steel beam, or both of these components, may be fully or partially propped during Construction Stages 3 and 4. The reactive forces in the props are effectively transferred onto the composite beam when the props are removed during Construction Stage 5. Therefore, in propped construction the dead load is either partly or fully carried by the composite beam, thereby reducing the stresses in the steel beam and the overall deflections compared with had the beam been built unpropped.

4. DESIGN MODELS

4.1 Representation of a Composite Beam at the Strength Limit State

When designing a composite beam for strength, the nominal dead load (G) and live load (Q) acting over the tributary area (A) (see Fig. 4.1(a)) are used to calculate the design load in accordance with Clause 4.1.4 of AS 2327.1. The effective section and design action effects (M^* and V^*) must be calculated in accordance with Clauses 5.2 and 5.3, respectively, and using an effective span (L_{ef}) determined in accordance with Appendix H of AS 2327.1 (see Fig. 4.1(a)). In the absence of the influence of vertical shear, the design moment capacity (ϕM_b) at a cross-section can be determined by considering equilibrium of the beam as a whole and the free bodies shown in Figs 4.1(b) and (c). Moment-shear interaction may reduce the design moment capacity as explained in Section 3.4, in which case the design moment capacity is represented by ϕM_{bv} (see Section 4.3).

The main features of the design model shown in Fig. 4.1 are:

(a) the beam is simply-supported at both ends;

(b) a "lightly-loaded" cantilever may exist at either end of the beam;

(c) the effective width of the concrete flange is determined, which makes allowance for the shear-lag effect in the slab;

(d) at cross-sections where design moment capacity is calculated, an effective portion of the steel beam section is determined, taking into account any effects of local buckling;

(e) the stress distribution at a cross-section at the strength limit state is determined using rectangular stress block theory; and

(f) when calculating the design moment capacity (ϕM_b) at a cross-section, the shear connectors on the side of the cross-section where nf_{ds} is minimum are assumed to be sufficiently ductile (see Section 4.2) to all sustain their design shear capacity f_{ds} together, provided the compressive force in the concrete $C = nf_{ds}$ does not exceed F_{cc}.

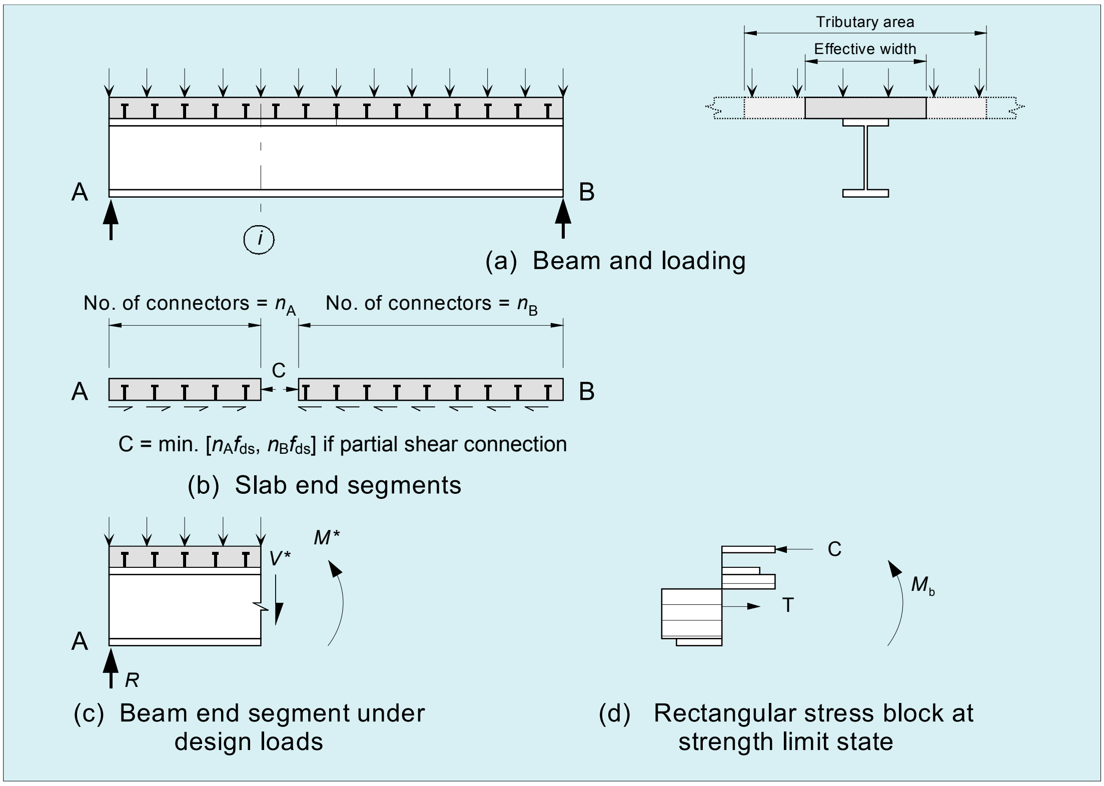


Figure 4.1 Representation of a Composite Beam Ignoring Moment-Shear Interaction at the Strength Limit State

4.2 Ductile Shear Connection Model

The general behaviour of shear connectors is described in Section 3.1. The ductile shear connection model used in the strength design method is shown in Fig. 4.2. It is assumed that shear connectors designed in accordance with Section 8 of AS 2327.1, and reinforced locally in accordance with Section 9 of the Standard have sufficient ductility for the assumption in Fig. 4.2 to be valid. Beam tests have shown that this is a valid assumption even at cross-sections a relatively long distance from the beam ends and with low values of degree of shear connection (β) [5]. The requirement of Clause 6.6.2(a) of AS 2327.1 that the degree of shear connection at the cross-section of maximum design bending moment (β_m) is not to be less than 0.5 is intended to reduce the demand for ductility of the shear connection.

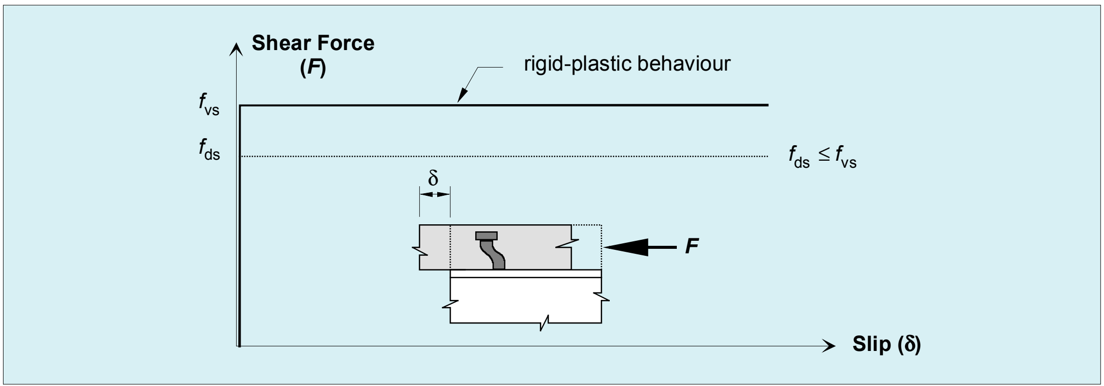


Figure 4.2 Ductile Shear Connection Model

4.3 Moment-Shear Interaction Model

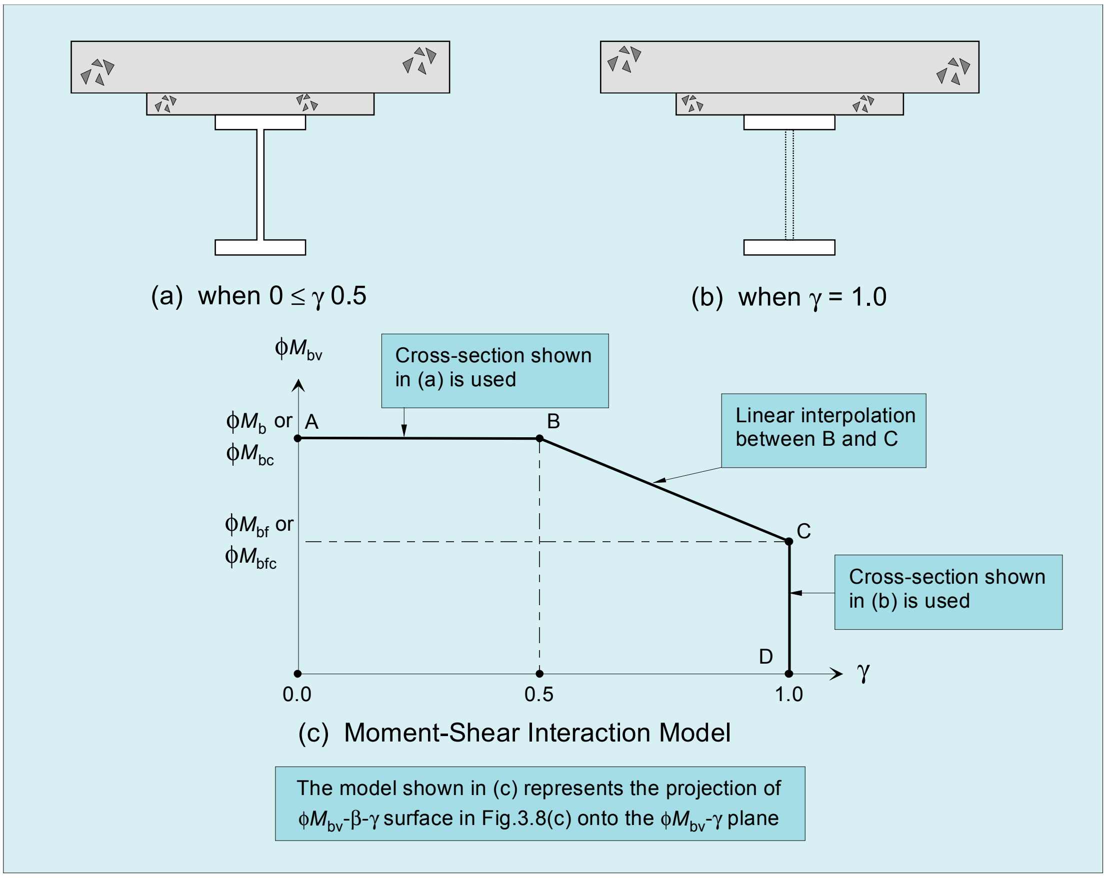


Figure 4.3 Moment-Shear Interaction Model

A detailed description of the moment-shear interaction behaviour is given in Sections 3.4 and 3.5. The basis of the design model used is given below.

(a) Unless it can be demonstrated that the concrete slab contributes to the vertical shear capacity, the web of the steel beam is assumed to carry the entire shear force.

(b) When the shear ratio (γ) is in the range $0 \leq \gamma \leq 0.5$, the nominal moment capacity (M_b or M_{bc}) of a cross-section is not affected by shear force, and the entire web of the effective portion of the steel beam is available to resist bending (i.e. between points A and B in Fig. 4.3(c)).

(c) When the shear ratio (γ) is equal to 1.0, the entire steel web is utilised carrying shear, and hence, the nominal moment capacity (M_{bf} or M_{bfc}) is calculated assuming the web is omitted (i.e. point C in Fig. 4.3(c)).

(d) Linear interpolation is used when the shear ratio (γ) is between 0.5 and 1.0 (i.e. between points B and C in Fig. 4.3(c)).

The moment-shear interaction diagram shown in Fig. 4.3(c) is a condensed two-dimensional form of the three-dimensional surface shown in Fig. 3.8.

5. DESIGN APPROACH

5.1 General

The purpose of this section is to explain the design approach adopted in this booklet. The actual design rules are referred to in Section 6, and cover the design for bending strength and vertical shear of simply-supported composite beams.

5.2 Limit State Requirements

The composite beam shall be designed so that at every transverse cross-section:

(a) the design vertical shear capacity (ϕV_u) is greater than or equal to the design vertical shear force (V^*), i.e. $\phi V_u \geq V^*$; and

(b) the design moment capacity (ϕM_{bv}) is greater than or equal to the design bending moment (M^*), both during construction and for the in-service condition, i.e. $\phi M_{bv} \geq M^*$.

The above requirements are deemed to be satisfied at every cross-section if they are shown to be satisfied at each of the relevant potentially critical cross-sections defined in Clause 6.3 of AS 2327.1.

5.3 Design Procedure Flowchart

It is assumed that the slab has been designed when the beam design is undertaken.

The design procedure should achieve the objectives stated in Section 5.1 while satisfying the limit state requirements stated in Section 5.2. The following two step-by-step procedures for strength design are given in AS 2327.1:

(a) a Simplified Procedure (Clause 6.2.3.2); and

(b) a General Procedure (Clause 6.2.3.3).

The Simplified Procedure is similar to the procedure in AS 2327.1–1980 which was essentially intended for designing prismatic beams with uniformly-distributed loading. The mid-span cross-section was designed to have complete shear connection, and no other cross-sections were checked for strength in bending except that the shear connector distribution might have to be adjusted if concentrated loads were present. Although the Simplified Procedure may not produce the most economical designs, it was included mainly to facilitate an easier transition to the General Procedure which is based on partial shear connection strength theory.

The General Procedure can also be used to design non-prismatic beams with non-uniform loads, including concentrated loads such as arises where primary beams support the ends of secondary beams. As explained in Section 4.2, it is a requirement that $\beta_m \geq 0.5$. The design approach on which the General Procedure is based is broadly summarised in the flowchart given in Fig. 5.1. Some of the most important steps of the design approach are described in subsequent sections.

5.4 Representation of Composite Beam and Loading

The first step in the process, when designing for strength, is to imagine the composite beam being designed, as separated from the rest of the floor system. The concrete slab will have already been designed to span between the steel beams in the frame, and its propping requirements, if any, will be known. The composite beam must be simply-supported, and consideration must be given to:

(a) determining the effective span (L_{ef}) (see Section 4.1);

(b) identifying the construction stages that may be critical (see Section 2); and

(c) determining the design loads relevant to construction and the in-service condition (see Section 4.1).

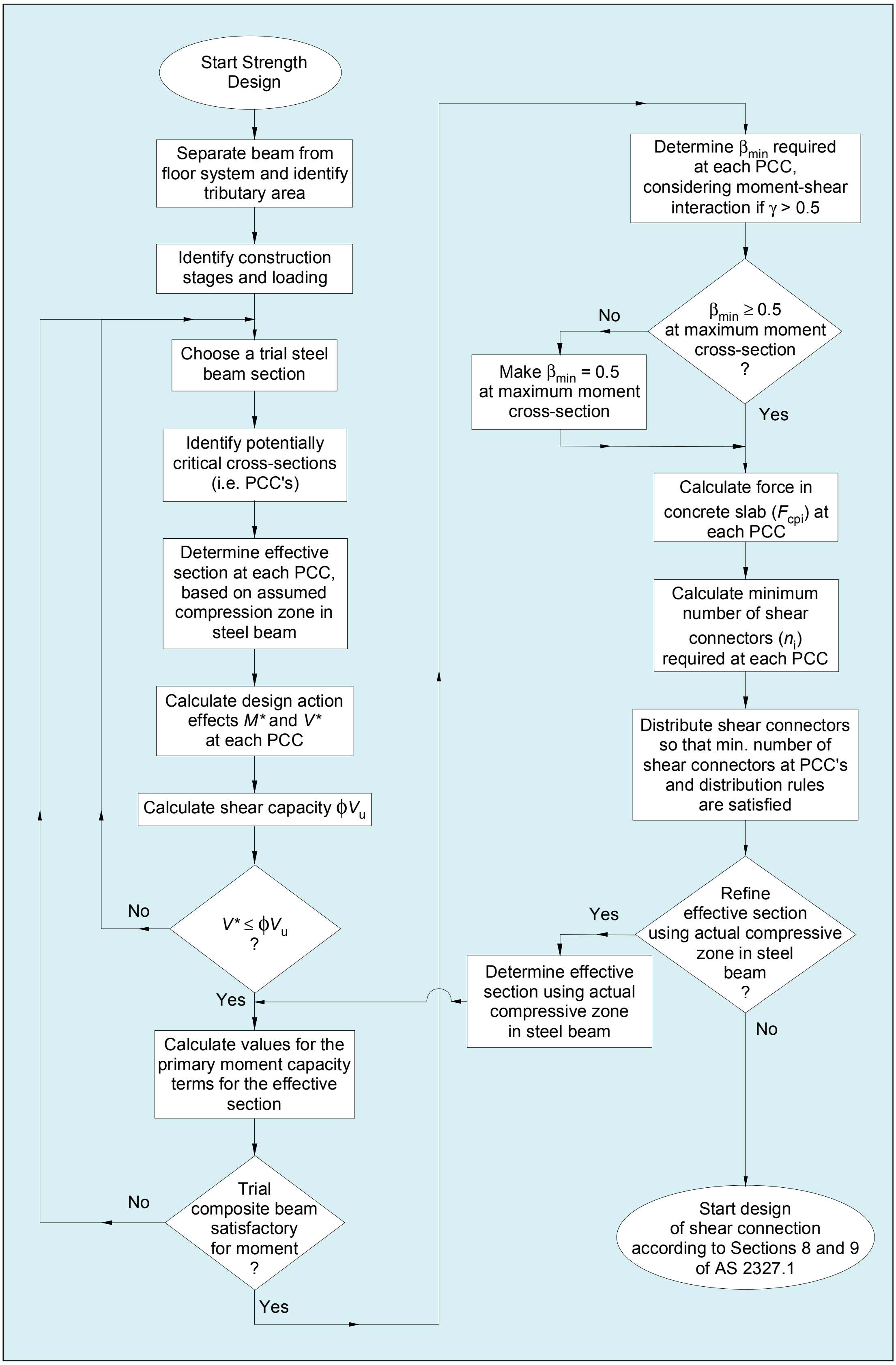


Figure 5.1 Design Approach used in General Procedure

5.5 Potentially Critical Cross-Sections (PCC's)

A simply-supported composite beam may fail due to formation of a plastic hinge within the span or due to shear failure in the end regions. A cross-section at which bending or shear failure occurs at the strength limit state is called a critical cross-section. Its location depends on a number of factors, such as the geometry of the beam, the material properties, the strength of the shear connection, and the loading configuration.

It is not possible to determine the exact location of the critical cross-section of a composite beam at the beginning of the design process, mainly because the shear connector distribution, and therefore the design moment capacity curve, is not known at this stage. Instead, the cross-sections that are likely to be critical are identified as "potentially critical cross-sections" or PCC's. Only the strength of these cross-sections must be checked during design. They are defined in Clause 6.3 of AS 2327.1, and are categorised as being potentially critical with respect to either bending (including moment-shear interaction) or shear (M^* effectively zero).

5.6 Effective Sections of a Composite Beam

Once the potentially critical cross-sections are identified, the designer needs to calculate their design moment capacity (ϕM_{bv}) and design shear capacity (ϕV_u). In order to do this, the effective section at each PCC must first be determined.

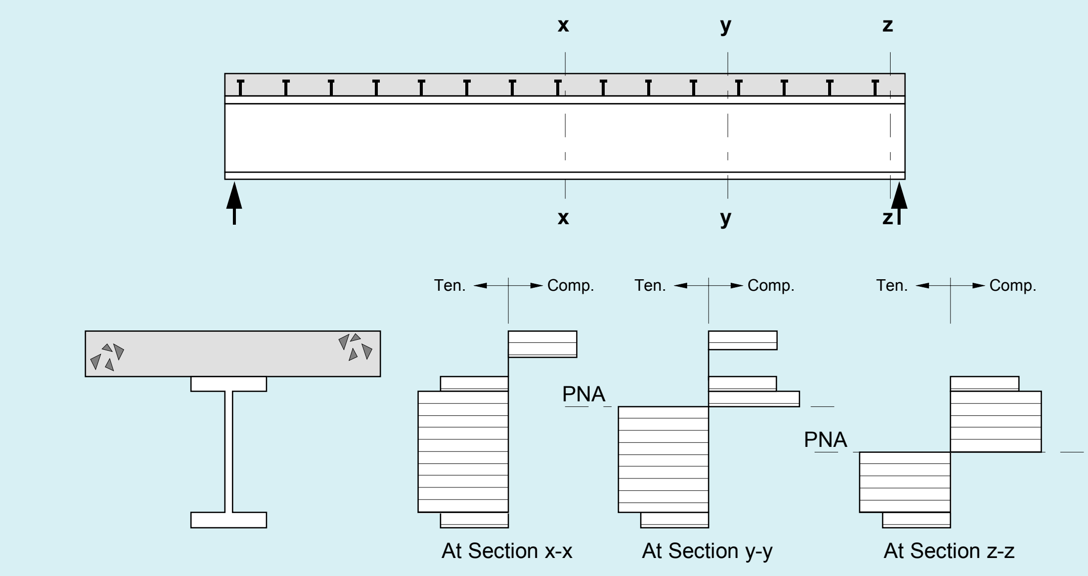
The concept of an effective section was described in Section 3.2, and a flowchart for calculating it is given in Fig. E2 of AS 2327.1. The calculation of the effective width of the concrete slab (b_{cf}) is straightforward. However, the process for determining the effective portion of the steel beam is more involved and may be iterative since the depth of the compressive zone in the steel beam at a PCC is not necessarily known initially. The way the depth of the compressive stress zone in the steel beam would vary along the length of a composite beam if successive cross-sections were critical is shown in Fig. 5.2. This is the approach taken to calculate a capacity curve using rectangular stress block theory (see Section 3.2). The depth of the compressive stress zone is greatest close to the supports where the degree of shear connection (β) is zero. Therefore, a conservative assessment of the effective section can be made assuming $\beta = 0$ (see right-hand branch in Fig. E2 of AS 2327.1). In cases where there is a change in the geometry or the material properties of either the steel beam or the concrete slab along the composite beam, more than one effective section will need to be determined initially.

Normally, it will not be necessary to refine the calculation of the effective section at a PCC once the strength of the shear connection is known better (i.e. when the value of β at the PCC is known better) and a more accurate assessment of the depth of the compressive stress zone can be made. Most hot rolled steel sections are compact and therefore fully effective even when $\beta = 0$.

When the compressive stress zone at the cross-section does not extend into the steel beam, the whole of the steel beam cross-section is assumed to be effective (see Clause 5.2.3.2 of AS 2327.1). This situation can only occur when there is complete shear connection, i.e. $\beta = 1.0$. Even for this condition, however, steel beams with slender sections are not allowed in AS 2327.1 (see Clause 1.2.2).

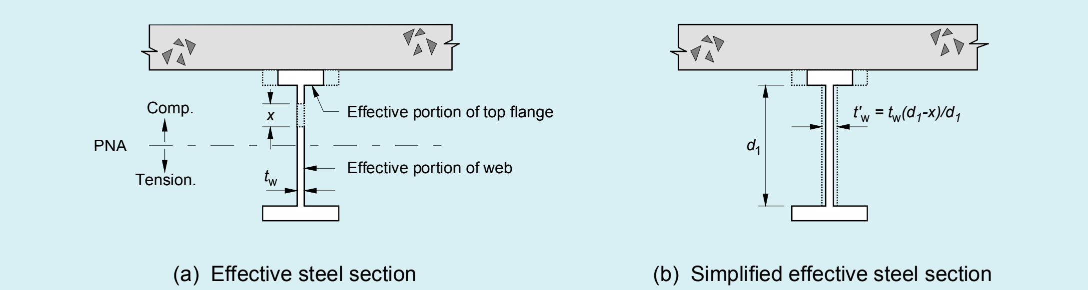
When part of the steel beam cross-section is in compression (which will always be the case when $\beta < 1.0$ and may occur when $\beta = 1.0$), the plate elements that are fully or partly in compression should be assessed for compactness in accordance with Clause 5.2.3.3 of AS 2327.1. Compact plate elements are considered fully effective. For non-compact plate elements, only the portion that is considered compact is included in the effective portion of the steel beam.

For commonly-used steel sections, the compressive stress zone at a cross-section will rarely extend into the bottom flange of the steel beam. This can occur, however, if a plate is welded to the underside of the bottom flange to strengthen the beam, in which case the compactness of the web may prove critical. A cross-section in which the top flange and the web are non-compact is shown in Fig. 5.3 (a). Then the effective portion of the steel beam may be simplified as shown in Fig. 5.3(b), so that the formulae given in Appendix D of AS 2327.1 for calculating the design moment capacity (ϕM_{bv}) can be used directly.



- The entire steel beam is in tension at Section x-x
- Part of the steel beam is in compression at Section y-y
- Plastic neutral axis at mid height at Section z-z for doubly-symetrical steel sections

Figure 5.2 Variation of Depth of Compressive Stress Zone in Steel Beam



- Steel top flange is non-compact only the width considered compact is effective
- Steel web is non-compact use effective portion only
- Effective portion of web is simplified to make it continuous over the entire depth
- Steel beam bottom flange in tension fully effective

Figure 5.3 Effective Portion of Steel Beam with Non-Compact Top Flange or Web

5.7 Design Action Effects at PCC's

It is a straightforward matter to calculate the design action effects (M^* and V^*) at the PCC's. As mentioned in Section 3.6, the design moment capacity of the cross-sections of a composite beam with compact cross-sections and ductile shear connection is independent of the sequence and method (i.e. propped or unpropped) of construction and does not need to be considered at the strength limit state. However, it is possible that propping will remain in place during Construction Stages 5 and 6, which will affect the calculation of the tributary area of the beam during this period.

The following steps are involved in the calculation of M^* and V^* at the PCC's.

(a) Determine the tributary area (A) for the beam, taking into account the effects of any propping to the slab.

(b) Determine the nominal loads for the relevant construction stages and the in-service condition.

(c) Calculate the design loads for the relevant construction stages and the in-service condition.

(d) Determine the effective span (L_{ef}).

(e) Calculate $M*$ and $V*$.

Information required for calculating the effective span (L_{ef}) and tributary area (A) is given in Appendix H of AS 2327.1.

5.8 Design Moment Capacity (ϕM_{bv}) versus Degree of Shear Connection (β) Relationship and Minimum Degree of Shear Connection (β_i) at a PCC

The design moment capacity (ϕM_{bv}) vs degree of shear connection (β) relationship at a PCC is required to determine the minimum degree of shear connection (β_i) necessary to resist the design bending moment ($M*$) at PCC (i). This relationship is specific to each effective section. As described in Section 3.5, ϕM_{bv} is a function of both β and γ. Bi-linear approximations to these relationships are considered quite accurate and can be used in design.

When $\gamma \le 0.5$

The design moment capacity (ϕM_{bv}) for this case is assumed to be unaffected by shear force, and therefore ϕM_b is a function of β only. The continuous and approximate bi-linear forms of the relationship between ϕM_b and β is shown in Fig. 5.4. The bi-linear approximation, which is used in design, can be used to calculate β_i as given below (see Clause 6.5.2 of AS 2327.1).

For $M* \le \phi M_s$

$$\beta_i = 0 \qquad (5.1)$$

For $\phi M_s < M* \le \phi M_{b.5}$

$$\beta_i = \frac{M* - \phi M_s}{2(M_{b.5} - \phi M_s)} \ge 0 \qquad (5.2)$$

For $\phi M_{b.5} < M* \le \phi M_{bc}$

$$\beta_i = \frac{M* + \phi M_{bc} - 2\phi M_{b.5}}{2(M_{bc} - \phi M_{b.5})} \ge 0 \qquad (5.3)$$

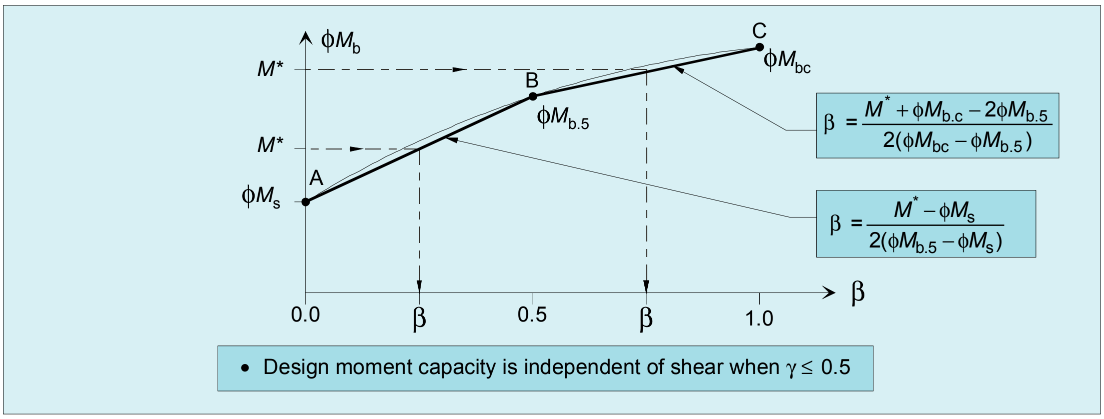
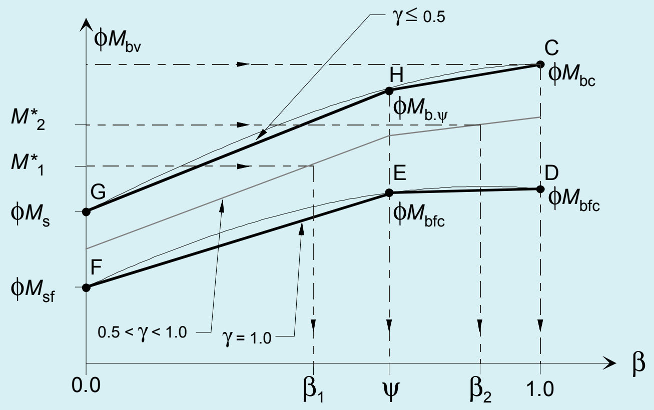


Figure 5.4 Relationship between ϕM_b and β for $\gamma \le 0.5$

When $0.5 < \gamma \le 1.0$

When $0.5 < \gamma \le 1.0$, the design moment capacity (ϕM_{bv}) is a function of both β and γ, as shown in Fig. 3.8. Several two-dimensional sections through the surface in Fig. 3.8 are shown in Fig. 5.5 along with the approximate bi-linear curves used in design. Curves GHC and FED correspond to the cases of $\gamma \le 0.5$ and $\gamma = 1.0$, respectively.



- Curve GHC is derived using full effective section
- Curve FED is derived using effective section without the web
- When $0 \leq \beta \leq \psi$ Interpolate using the moment capacities corresponding to G, H, E and F
- When $\beta > \psi$ Interpolate using the moment capacities corresponding to H, C, D and E

Figure 5.5 Relationship between ϕM_{bv} and β for $0.5 < \gamma \leq 1.0$

Curve GHC corresponds to the entire effective section being available to resist bending, while curve FED is derived assuming that the steel web is completely utilised resisting vertical shear and is not available to resist bending (see Section 4.3). With the steel web omitted, the tensile capacity of the assumed steel section is less and, therefore, curve FED reaches a peak value of ϕM_{bfc} at $\beta = \psi$, where $\psi < 1.0$. Point H is used as an intermediate point for the bi-linear approximation to curve GHC since it has the same β value as point E.

For a given value of M^* and γ, the value of β_i can be found by linear interpolation between:

(a) points G, H, F and E when $0 \leq \beta \leq \psi$; and

(b) points H, C, E and D when $\beta > \psi$.

The interpolation equations (Eqs 6.5.3(1) and 6.5.3(2) of AS 2327.1) are given below:

For $0 \leq \beta_i \leq \psi$

$$\beta_i = \frac{[M^* - (2\gamma - 1)\phi M_{sf} - 2(1-\gamma)\phi M_s]\psi}{(1-2\gamma)\phi M_{sf} + (2\gamma - 1)\phi M_{bfc} - 2(1-\gamma)\phi M_s + 2(1-\gamma)\phi M_{b.\psi}} \geq 0 \tag{5.4}$$

For $\psi < \beta_i$

$$\beta_i = \frac{(1-\psi)\left[M^* - 2(1-\gamma)\phi M_{b.\psi} - (2\gamma - 1)\phi M_{bfc}\right]}{2(1-\gamma)(\phi M_{bc} - \phi M_{b.\psi})} \tag{5.5}$$

If the value of β calculated from the above equations is greater than 1.0, it indicates that the section is inadequate.

At cross-section(s) of maximum design bending moment (M^*), it is a requirement that β_i is not be less than 0.5 (see Section 4.2).

Design Aids

Values of the design moment capacities ϕM_s, $\phi M_{b.\psi}$, ϕM_{bc}, ϕM_{sf} and ϕM_{bfc} (i.e. points G, H, C, F and E or D in Fig. 5.5) and $\phi M_{b.5}$ (i.e. point B in Fig. 5.5), and values of ψ required to use in Eqs 5.1 to 5.4 are tabulated in the Composite Beam Design Handbook [2] for commonly used composite beam cross-sections.

5.9 Compressive Force in Concrete ($F_{cp.i}$) at a PCC

The values of β_i at each PCC, calculated as described in Section 5.8, can then be used to determine the corresponding compressive forces in the concrete flange, using the following relationship.

$$F_{cp.i} = \beta_i F_{cc} \tag{5.6}$$

The number of shear connectors provided on either side of the PCC should be able to develop the force F_{cp} in the concrete flange.

6. DESIGN RULES

6.1 General

Simply-supported composite beams shall be designed for strength in accordance with Section 6 and related sections of AS 2327.1. The effective section of the beam and the design action effects for strength shall be determined in accordance with Section 5 of AS 2327.1.

6.2 Design Objectives

The objective of Section 6 of AS 2327.1 is that beams of adequate strength, as defined in Clause 3.1.1 of the Standard, are designed to resist the design load. The composite beam shall also be designed for stability, serviceability, durability, fire resistance and for any other relevant design criteria, as stated in Clause 3.1.2 of AS 2327.1.

Note: This booklet covers design for bending strength and vertical shear only. Major aspects of the design have been explained in preceding sections. The normal design procedure to follow is described in Section 5.

7. WORKED EXAMPLES

7.1 General

The steps of the Simplified and General Procedures of strength design are given in Clauses 6.2.3.2 and 6.2.3.3 of AS 2327.1, respectively. These steps are also outlined in the flowchart given in Section 5.3 (see Fig. 5.1). The calculations involved in the steps that are covered within the scope of this booklet are demonstrated in this section using worked examples. Each sub-section covers one step in the design procedure and includes stand-alone examples to cover different design situations. These worked examples supplement the detailed design examples given in references [2] and [5].

In some examples it has been necessary to choose values for key dimensions of the profiled steel sheeting, e.g. h_r, s_r, etc. which do not apply to all the profiled steel sheeting products. However, the calculation procedures remain unchanged.

The index to the worked examples is given in Table 7.1.

Table 7.1 Index to Worked Examples

7.2 Identification of Potentially Critical Cross-Sections (Clause 6.3, AS 2327.1)

The rules for identification of potentially critical cross-sections (PCC's) are given in Clause 6.3 of AS 2327.1, and the application of the rules is demonstrated using the examples given below.

Example 7.2-1 PCC's in a Prismatic Beam with a Uniformly-Distributed Load

Determine the positions of potentially critical cross-sections (PCC's) in the prismatic beam shown in Fig. 7.1 for which $M_{bc} \leq 2.5 M_s$.

(Note: This condition arises only when the steel beam used in a composite beam is relatively small in comparison to the slab.)

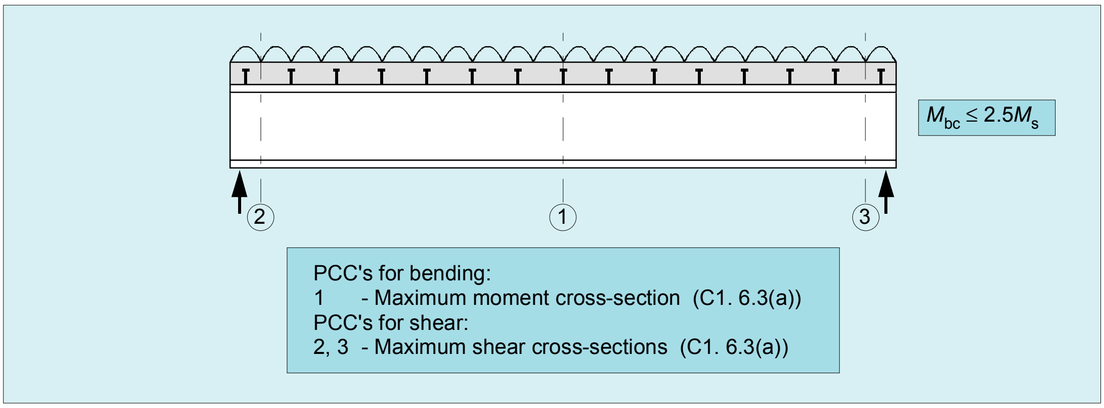


Figure 7.1 PCC's in a Prismatic Beam where $M_{bc} \le 2.5\ M_s$ – Example 7.2-1

Solution

As shown in Fig. 7.1, the critical cross-section for bending occurs at mid-span. Therefore, one PCC for bending and two PCC's for shear are considered (see Fig. 7.1).

This beam can be designed using either the Simplified Procedure given in Clause 6.2.3.2 of AS 2327.1 or the General Procedure given in Clause 6.2.3.3.

Example 7.2-2 PCC's in a Beam where $M_{bc} > 2.5\ M_s$

Determine the positions of potentially critical cross-sections (PCC's) in the prismatic beam shown in Fig. 7.2.

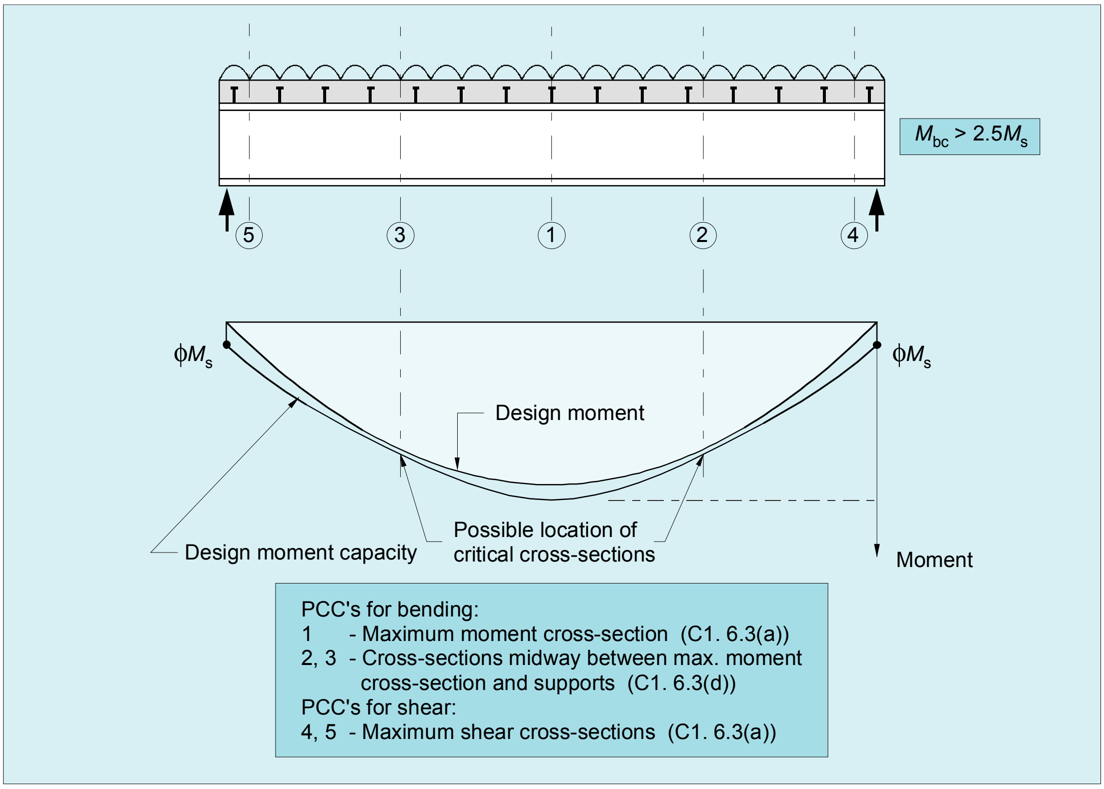


Figure 7.2 PCC's in a Prismatic Beam where $M_b > 2.5\ M_s$ - Example 7.2-2

Solution

As shown by the bending moment diagram in Fig. 7.2, the critical cross-sections for bending in the beam occur at locations approximately midway between the maximum moment cross-section and the supports. Therefore, in accordance with Clause 6.3(d) of AS 2327.1, PCC's 2 and 3 midway between the section of maximum design bending moment and the adjacent ends of the beam, are also considered in addition to PCC 1.

Example 7.2-3 PCC's in a Prismatic Beam with Concentrated Loads

Determine the positions of potentially critical cross-sections (PCC's) for the beam shown in Fig. 7.3.

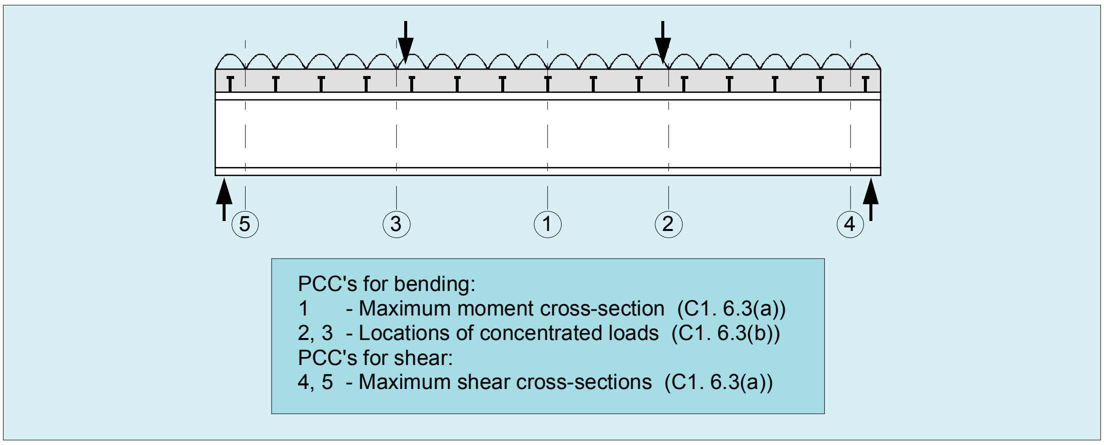


Figure 7.3 PCC's in a Prismatic Beam with Concentrated Loads - Example 7.2-3

Solution

The PCC's identified are shown in Fig. 7.3. Note that the PCC's 2 and 3 at the locations of concentrated loads have been marked next to the load on the side where the shear force is higher.

Example 7.2-4 PCC's in a Non-Prismatic Beam

Determine the positions of potentially critical cross-sections (PCC's) in the non-prismatic beam shown in Fig. 7.4.

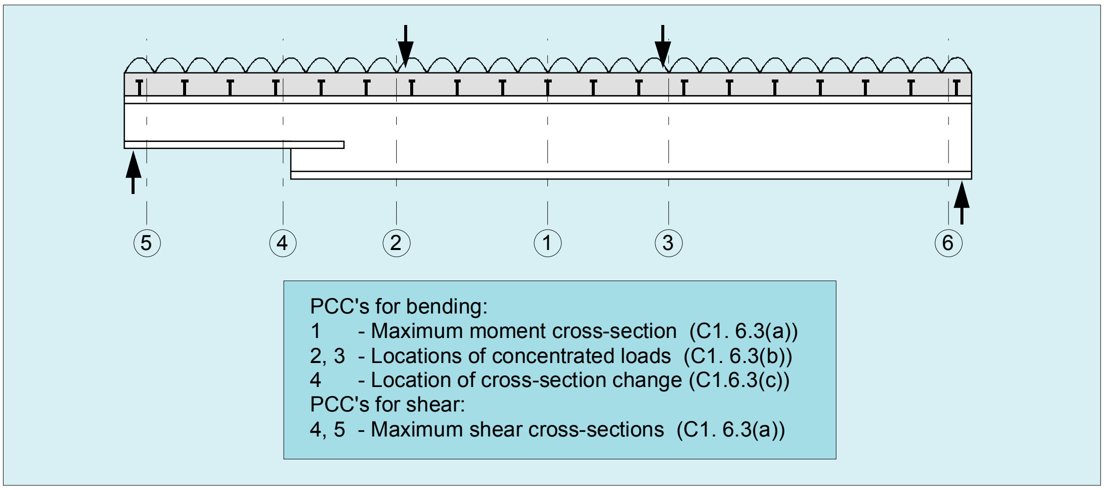


Figure 7.4 PCC's in a Non-Prismatic Beam - Example 7.2-4

Solution

The PCC's are shown in Fig. 7.4.

7.3 Calculation of Effective Section (Clause 5.2, AS 2327.1)

The design rules for calculation of effective section are given in Clause 5.2 of AS 2327.1. The effective width of the concrete flange is determined in accordance with Clause 5.2.2, and the effective portion of the steel beam is determined in accordance with Clause 5.2.3.

Application of the rules given in Clauses 5.2.2 and 5.2.3 are demonstrated using the examples given below.

Example 7.3-1 Calculation of Effective Width of Concrete Flange for an Internal Beam

Determine the effective width of the concrete flange for the internal composite beam shown in Fig. 7.5. The sheeting ribs are perpendicular to the longitudinal axis of the beam.

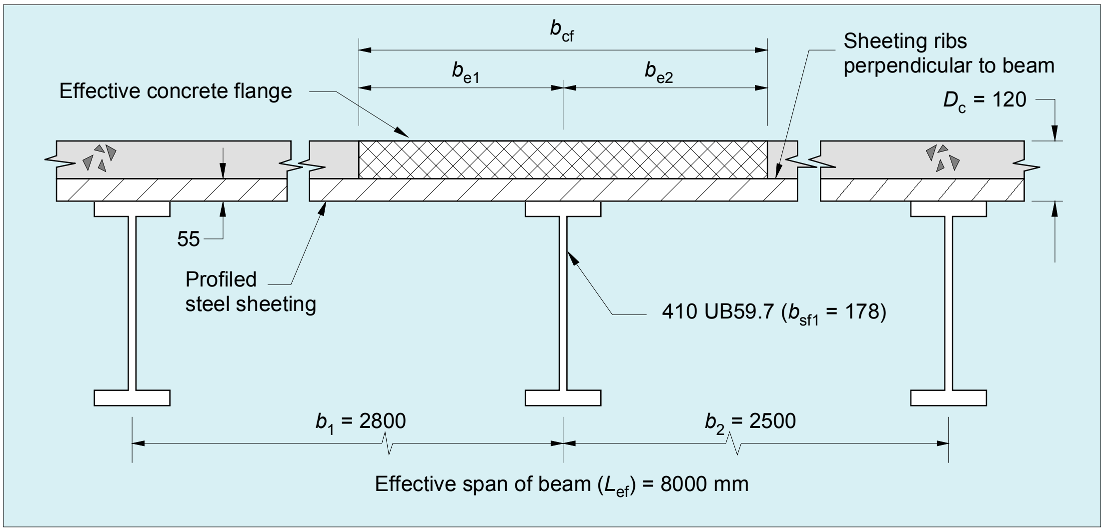


Figure 7.5 Dimensions and other Data for Example 7.3-1

Solution

Using Clause 5.2.2.1, AS 2327.1,

$$b_{e1} = \min.\left|\frac{L_{ef}}{8}, \frac{b_1}{2}, \frac{b_{sf1}}{2} + 8D_c\right|$$

$$= \min.\left(\frac{8000}{8}, \frac{2800}{2}, \frac{178}{2} + 8 \times 120\right)$$

$$= \min.(1000, 1400, 1049) = 1000 \text{ mm}$$

Similarly, $$b_{e2} = \min.\left(\frac{8000}{8}, \frac{2500}{2}, \frac{178}{2} + 8 \times 120\right) = 1000 \text{ mm}$$

Therefore, $$b_{cf} = b_{e1} + b_{e2} = 2000 \text{ mm}$$

$$= \underline{\textbf{2000 mm}}$$

Since sheeting ribs are perpendicular to the longitudinal axis of the steel beam, only the portion of the slab above the ribs is considered effective.

Note that the value of b_{cf} calculated above is independent of the angle between the sheeting ribs and the longitudinal axis of the beam.

Example 7.3-2 Calculation of Effective Width of Concrete Flange for an External Beam

Determine the effective width of the concrete flange for the external composite beam shown in Fig. 7.6. The acute angle (θ) between the sheeting ribs and the steel beam is 45°. This example demonstrates the calculation of b_{cf} for an external beam as well as the calculation of λ.

Solution

Using Clause 5.2.2.1, AS 2327.1,

$$b_{e1} = \min.\left(\frac{L_{ef}}{8}, \frac{b_1}{2}, \frac{b_{sf1}}{2} + 8D_c\right)$$

$$= \min.\left(\frac{8000}{8}, \frac{2800}{2}, \frac{178}{2} + 8 \times 120\right)$$

$$= \min.(1000, 1400, 1049) = 1000 \text{ mm}$$

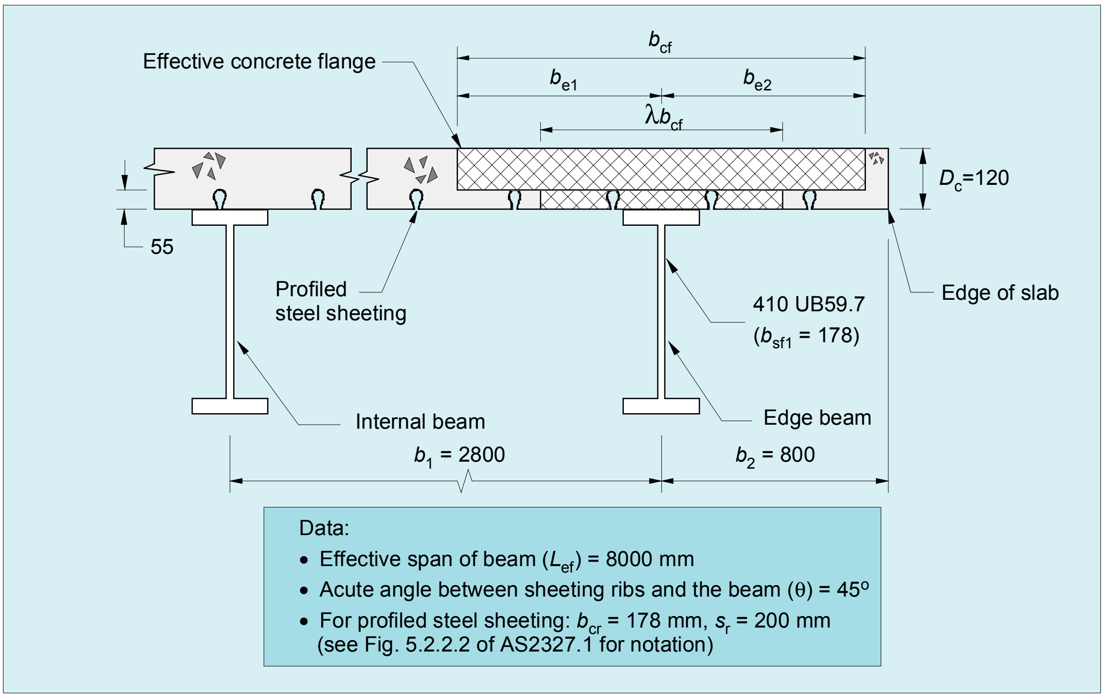


Figure 7.6 Dimensions and other Data - Example 7.3-2

Similarly, $$b_{e2} = \min.\left|\frac{L_{ef}}{8}, b_2, \frac{b_{sf1}}{2} + 6D_c\right|$$

$$= \min.\left(\frac{8000}{8}, 800, \frac{178}{2} + 6 \times 120\right)$$

$$= \min.(1000, 800, 809) = 800 \text{ mm}$$

Therefore, $$b_{cf} = b_{e1} + b_{e2} = 1000 + 800 = 1800 \text{ mm}$$

$$= \underline{\mathbf{1800\ mm}}$$

Since the acute angle (θ) between the sheeting ribs and the beam longitudinal axis is 45°, a portion of the concrete within the depth of the ribs is also effective (see Clause 5.2.2.2, AS 2327.1).

From Eq. 5.2.2.2(1) of AS 2327.1,

$$\lambda = (b_{cr} \cos^2 \theta) / s_r$$

$$= \frac{178 \times \cos^2(45)}{200} = 0.45$$

Therefore, $$\lambda b_{cf} = 0.45 \times 1800$$

$$= \underline{\mathbf{810\ mm}}$$

The effective concrete flange of the external beam is shown in Fig. 7.6.

Note that for commonly available Australian profiled steel sheeting products (i.e. BONDEK II, COMFORM and CONDECK HP), the value of λ equals 1.0 when $\theta = 0$ and 0.0 when

Example 7.3-3 Calculation of Effective Portion of Steel Beam when β is Unknown

The dimensions of a composite beam cross-section, where a fabricated 3-plate girder is used as the steel section, are given in Fig. 7.7(a). The shear connection (i.e. value of β) at the PCC is not known at this stage of the design. Determine the effective portion of the steel beam.

Solution

The position of the plastic neutral axis in the cross-section cannot be determined since the value of β is unknown. The most conservative approach is to assume $\beta = 0$, i.e. beam is acting non-compositely.

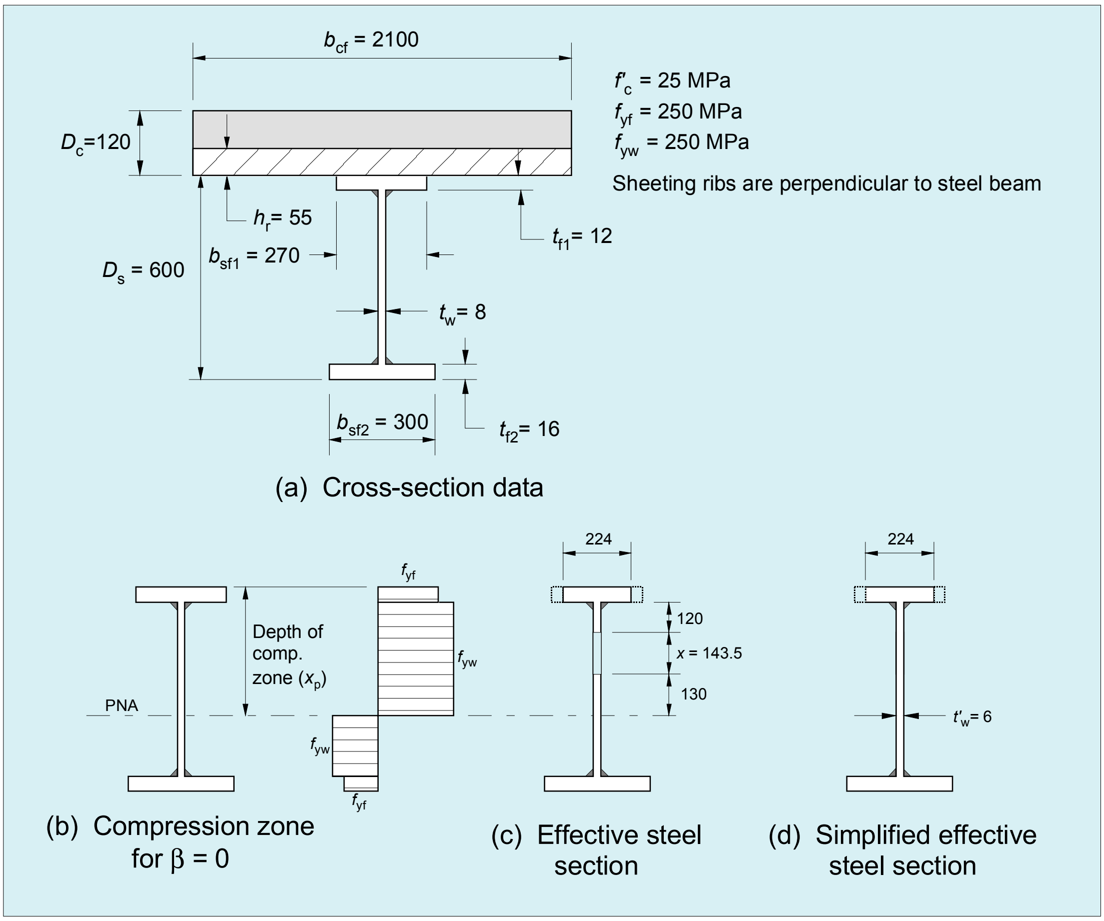


Figure 7.7 Data and Solution - Example 7.3-3

Using Fig. 7.6(b), and considering the equilibrium of forces in the cross-section,

$$b_{sf1}t_{f1}f_{yf} + (x_p - t_{f1})t_w f_{yw} = (D_s - x_p - t_{f2})t_w f_{yw} + b_{sf2}t_{f2}f_{yf}$$

Substituting the values for cross-section dimensions and yield stresses given in Fig. 7.7(a) and solving for x_p,

$$x_p = 395.5 \text{ mm}$$

Therefore, the plastic neutral axis falls within the web of the steel beam, and the compactness of the top flange outstands and the web should be determined.

Flange Outstand:

$$\lambda_e = \left|\frac{b}{t_{f1}}\right|\sqrt{\frac{f_y}{250}} = \left(\frac{(270-8)/2}{12}\right)\sqrt{\frac{250}{250}} = 10.9$$

From Table 5.1 of AS 2327.1,

$$\lambda_{ep} = 9 \quad \text{and}$$
$$\lambda_{ey} = 16$$

Since $\lambda_{ep} < \lambda_e < \lambda_{ey}$, the top flange of the steel beam is non-compact.

$$\text{Width of the compact outstand} = \lambda_{ep}t_{f1}\sqrt{\frac{250}{f_{yf}}} = 9 \times 12 \times 1 = 108$$

$$\text{Width of the compact portion} = 108 \times 2 + 8 = 224 \text{ mm}$$

Web:

$$\lambda_e = \left|\frac{b}{t_{f1}}\right|\sqrt{\frac{f_y}{250}} = \left(\frac{600-12-16}{8}\right)\sqrt{\frac{250}{250}}$$

$$= 71.5$$

Using Table 5.1 of AS 2327.1,

$$r_p = \frac{(395.5-12)}{(600-12-16)}$$

$$= 0.67$$

$$\therefore \quad \lambda_{ep} = \frac{111}{(4.7r_p-1)} = \frac{111}{(4.7\times 0.67-1)}$$

$$\lambda_{ep} = 51.7$$

Determine the elastic neutral axis depth for the steel section (x_e) as shown below:

$$\text{Total area of steel cross-section} = 270\times 12+572\times 8+300\times 16$$

$$= 12616\ \text{mm}^2$$

$$x_e = (270\times 12\times 6+572\times 8\times 298+300\times 16\times 592)/12616$$

$$= 334.9\ \text{mm}$$

$$\therefore \quad r_e = \frac{(334.9-12)}{(600-12-16)}$$

$$= 0.56$$

Using Table 5.1 of AS 2327.1,

$$\lambda_{ey} = \frac{322}{(3.6r_e+1)} = \frac{322}{(3.6\times 0.56+1)}$$

$$= 106.8$$

Since $\lambda_{ep} < \lambda_e < \lambda_{ey}$, the web of the steel beam is non-compact.

Using Fig. 5.2.3.3(a) of AS 2327.1,

Depth of the effective web in compressive region

$$= 15t_w\sqrt{\frac{250}{f_y}} = 15\times 8\sqrt{\frac{250}{250}} = 120\ \text{mm}$$

$$\therefore \quad x = x_p - t_{f1} - 2\times 120$$

$$= 395.5-12-2\times 120$$

$$= 143.5\ \text{mm}$$

The effective steel section is shown in Fig. 7.7(c).

From Fig. 5.2.3.3(b) of AS 2327.1,

$$t_w' = t_w(d_1 - x)/d_1 = 8.0(572-143.5)/572$$

$$= 6.0\ \text{mm}$$

The simplified effective steel section is shown in Fig. 7.7(d).

Example 7.3-4 Calculation of Effective Portion of Steel Beam when β <1.0

Determine the effective portion of the steel beam for the cross-section shown in Fig. 7.7(a) when $\beta = 0.8$. *(Note: In Example 7.3-3, the effective portion of the steel beam for the same cross-section was determined approximately, assuming β = 0.)*

Solution

Since $\beta < 1.0$, at least a portion of the steel beam top flange will be in compression. Therefore, the effective width of the top flange can be calculated as shown in Example 7.3-3.

From Example 7.3-3,

Effective width of top flange = 224 mm

Determine F_{cc} using the formulae given in Paragraph D2.3.2 of AS 2327.1, assuming the entire area of the steel beam web and bottom flange are effective.

Using Eq. D2.3.2,

$$F_{st} = 3016 \text{ kN}$$
$$F_{c1} = 2901 \text{ kN}$$
$$F_{c2} = 0 \text{ kN}$$
$$F_{scf} = 672 \text{ kN}$$

Since $(F_{c1} + F_{c2}) < F_{st} \leq (F_{c1} + F_{c2} + 2F_{scf})$, Case 3 in Paragraph D2.3.2(c) is applicable. Therefore,

$$F_{cc} = F_{c1} + F_{c2} = 2901 \text{ kN}$$

Since $\beta = 0.8$,

$$F_{cp} = \beta F_{cc} = 0.8 \times 2901$$
$$= 2321 \text{ kN}$$

Calculate the plastic neutral axis corresponding to β = 0.8, using Paragraph D2.3.3 of AS 2327.1.

From Eq. 2.3.3(3),

$$F_{sc} = F_{st} - F_{cp}$$
$$= 3016 - 2321 = 695$$

Since $F_{sc} < 2F_{scf}$, from Eq. D2.3.3(6)

$$d_h = D_c + t_{f1} F_{sc} / (2F_{scf})$$
$$= 120 + 12 \times \frac{695}{2 \times 672}$$
$$= \underline{\mathbf{126.2\ mm}}$$

The plastic neutral axis falls within the steel beam top flange. Therefore, the bottom flange and the entire web of the steel section are effective.

The effective section of the beam for $\beta = 0.8$ is shown in Fig. 7.8. Note that the plastic neutral axis now falls within the steel beam top flange, and therefore the entire web is effective.

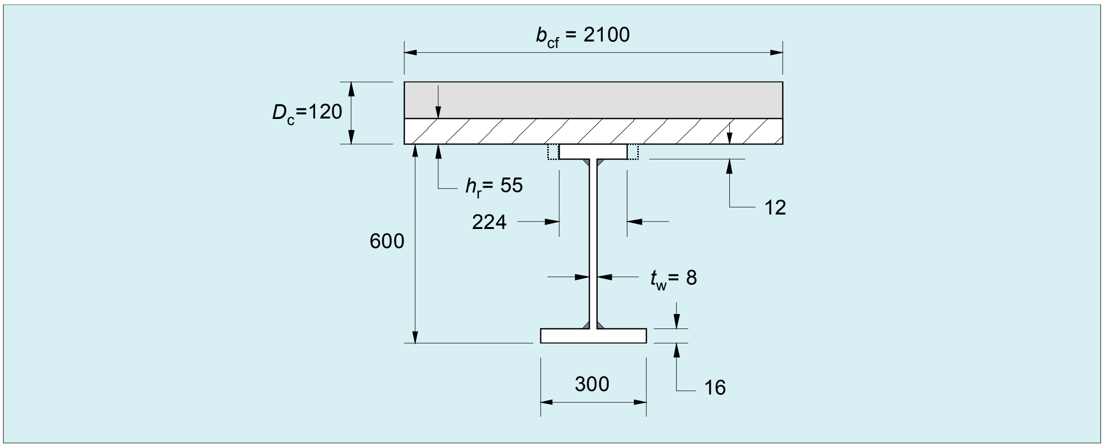


Figure 7.8 Effective Section for β = 0.8 - Example 7.3-4

7.4 Calculation of Design Action Effects (Clause 5.3, AS 2327.1)

Example 7.4-1 Calculation of Design Action Effects

The layout of a composite floor structure is shown in Fig. 7.9. Determine the design action effects for the secondary beam (Beam A) for the in-service condition.

Solution

The solution involves the calculation of the effective span, tributary area, design actions (i.e. loads) and design action effects (i.e. M^* and V^*).

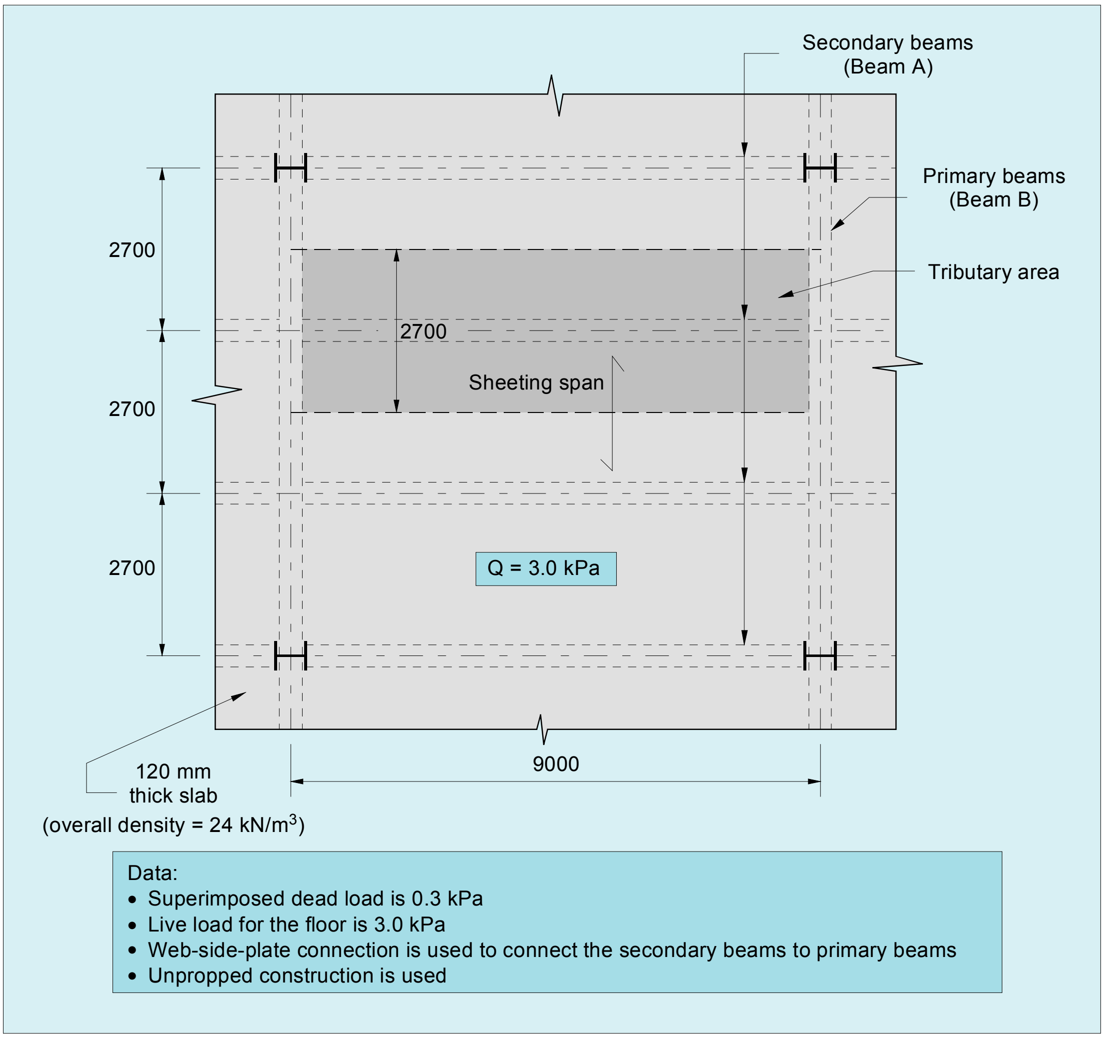


Figure 7.9 Floor Layout and Details - Example 7.4-1

The secondary beams are connected to the primary beams using the web-side-plate connection. Therefore, the support reaction can be assumed to pass through the centre-line of the primary beam cross-section, as shown in Fig. H1(d) of AS 2327.1. Therefore,

$$\text{Effective span } (L_{ef}) = 9.0 \text{ m}$$

The tributary area is determined using Paragraph H2 and Fig. H2 of AS 2327.1.

Loading for In-Service Condition:

Assume 0.7 kN/m for the self weight of the steel beam.

$$\text{Self weight} = 0.7 + 0.12 \times 2.7 \times 24 = 8.5 \text{ kN/m}$$

$$\text{Superimposed dead load } (G_{sup}) = 0.3 \times 2.7 = 0.81 \text{ kN/m}$$

$$\text{Dead load (G)} = 8.5 + 0.81 = 9.31 \text{ kN/m}$$

$$\text{Live load (Q)} = 3.0 \times 2.7 = 8.1 \text{ kN/m}$$

$$\text{Design load } (W) = 1.25G + 1.5Q = 1.25 \times 9.31 + 1.5 \times 8.1$$

$$= 23.8 \text{ kN/m}$$

Design Action Effects:

$$\text{Design moment at mid-span } (M^*) = 23.8 \times 9^2 / 8$$

$$= \underline{\mathbf{241 \text{ kNm}}}$$

$$\text{Design shear force at support } (V^*) = 23.8 \times 9 / 2$$

$$= \underline{\mathbf{107.1 \text{ kN}}}$$

It may be necessary to check the strength of the beam for the construction stages defined in Paragraph F1 of AS 2327.1. The design action effects for construction stages should be determined taking into account the appropriate construction loads. The minimum nominal loads for construction are given in Paragraph F2 of AS 2327.1.

7.5 Calculation of Design Vertical Shear Capacity (ϕV_u) (Clause 6.4.1, AS 2327.1)

Example 7.5-1 Calculation of Design Vertical Shear Capacity

Calculate the design vertical shear capacity for the composite beam cross-section shown in Fig. 7.10.

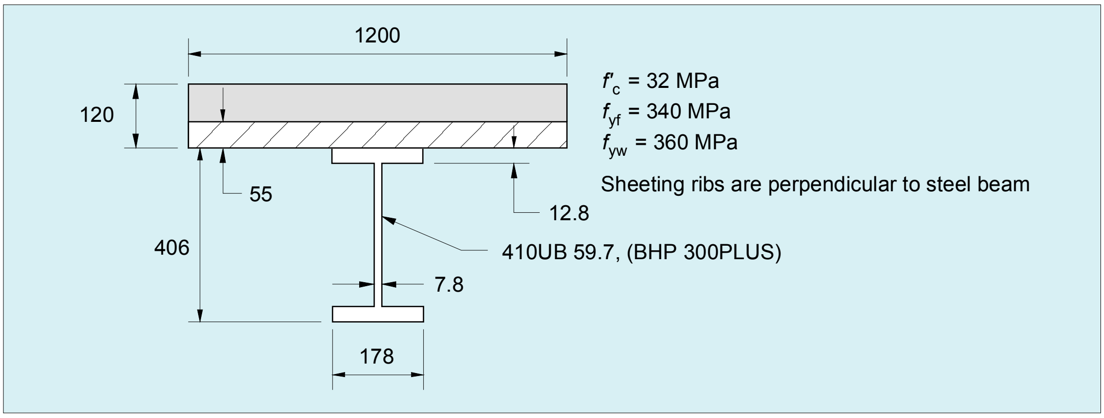


Figure 7.10 Composite Beam Cross-section - Examples 7.5-1 and 7.6-1

Solution

Assume that only the steel beam is effective in resisting shear.

As specified in Clause 6.4.1 of AS 2327.1, the design vertical shear capacity is calculated in accordance with Clause 5.11 of AS 4100.

Since the steel beam is a universal beam section (300PLUS®), the depth of the web is taken as the overall depth of the steel beam (D_s). (Note: The depth of the web is taken as the clear depth between the flanges when a fabricated 3-plate girder is used as the steel section.)

$$\left|\frac{D_s}{t_w}\right|\sqrt{\frac{f_{yw}}{250}} = \left|\frac{406}{7.8}\right|\sqrt{\frac{360}{250}} = 62.5 < 82$$

Therefore, the web is compact for shear and,

$$\begin{aligned}\phi V_u &= \phi V_w \\ &= 0.9 \times 0.6 f_w D_s t_w \\ &= 0.9 \times 0.6 \times 320 \times 406 \times 7.8 / 1000 \\ \phi V_u &= \underline{\mathbf{547\ kN}}\end{aligned}$$

The method given in Clause 5.11 of AS 4100 also covers the cases of steel beams with non-compact webs and non-uniform shear distribution.

7.6 Calculation of Design Moment Capacity (ϕM_b) versus Degree of Shear Connection (β) Relationship (Clause 6.4.2, AS 2327.1)

Example 7.6-1 Calculation of ϕM_b vs β when $\gamma \le 0.5$

Determine the ϕM_b versus β relationship for the composite beam cross-section shown in Fig. 7.10, when the shear ratio (γ) is less than or equal to 0.5.

Solution

The solution involves calculation of the design moment capacities ϕM_s, $\phi M_{b.5}$ and ϕM_{bc}, and then determining the equations describing the relationship between ϕM_b and β. Moment-shear interaction is not relevant since $\gamma \le 0.5$.

The design moment capacities ϕM_s, $\phi M_{b.5}$ and ϕM_{bc} can be determined using: the formulae given in Paragraph D2 of AS 2317.1; the design tables given in Composite Beam Design Handbook [2]; or the software COMPBEAM™.

From Table A4, Composite Beam Design Handbook [2],

$$\phi M_s = 323 \text{ kNm}$$
$$\phi M_{b.5} = 516 \text{ kNm}$$
$$\phi M_{bc} = 590 \text{ kNm}$$

Using Eqs. D2.2(1) and D2.2(2) of AS 2327.1,

for $0 < \beta \le 0.5$

$$\begin{aligned} \phi M_b &= (1-2\beta)\phi M_s + 2\beta\phi M_{b.5} \\ &= 323(1-2\beta) + 516 \times 2\beta \\ \phi M_b &= 323 + 386\beta \end{aligned}$$

and, for $0.5 < \beta < 1.0$

$$\begin{aligned} \phi M_b &= (2\beta-1)\phi M_{bc} - 2(\beta-1)\phi M_{b.5} \\ &= 590(2\beta-1) - 2 \times 516(\beta-1) \\ \phi M_b &= 442 + 148\beta \end{aligned}$$

The two relationships are shown in Fig. 7.11.

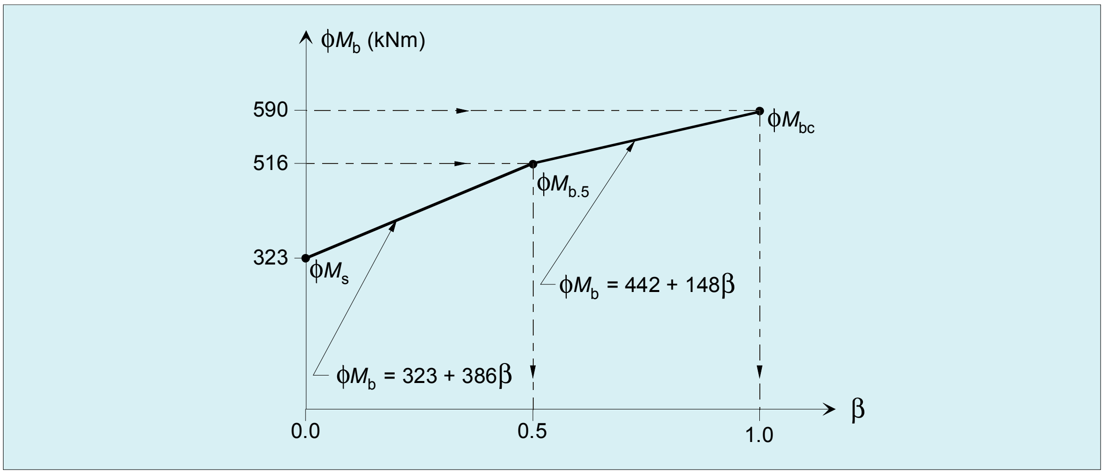


Figure 7.11 ϕM_b vs β Relationship - Example 7.6-1

Example 7.6-2 Calculation of ϕM_{bv} vs β Relationship when $\gamma > 0.5$

Determine the ϕM_{bv} versus β relationship for the composite beam cross-section shown in Fig. 7.10, when the shear ratio (γ) is greater than 0.5.

Solution

The solution involves the calculation of the design moment capacities ϕM_s, ϕM_{sf}, $\phi M_{b.y}$, ϕM_{bfc} and ϕM_{bc}, and then determining the equations describing the relationship between ϕM_{bv} and β.

From Table A4, Composite Beam Design Handbook [2],

ϕM_s =	323 kNm	ϕM_{sf} =	242 kNm
$\phi M_{b.\psi}$ =	541 kNm	ϕM_{bfc} =	372 kNm
ϕM_{bc} =	590 kNm	ψ =	0.64

The bi-linear relationships between ϕM_b and ϕM_{bf} vs β are shown in Fig. 7.12. For a given value of β, the value of ϕM_{bv} can be found by linearly interpolating between the two bi-linear curves.

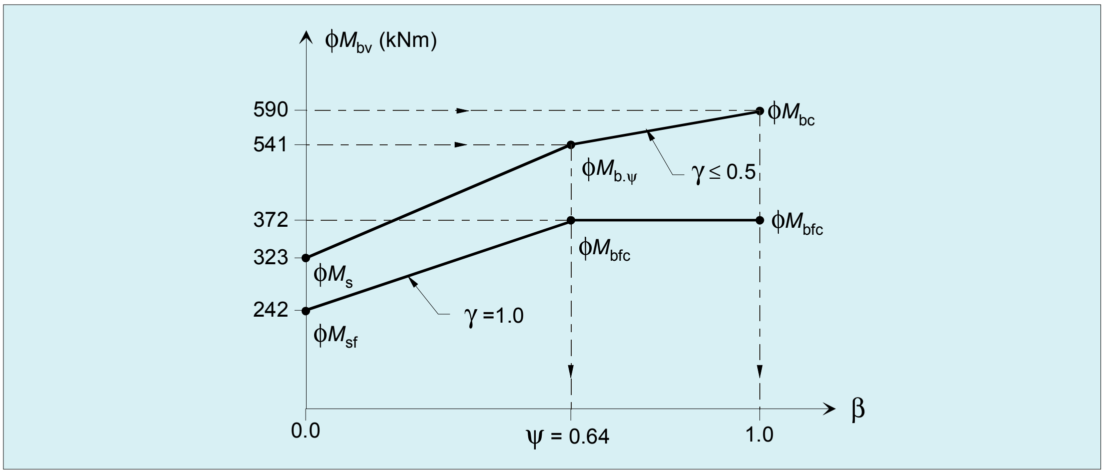


Figure 7.12 ϕM_b and ϕM_{bf} vs β Relationships - Example 7.6-2

7.7 Calculation of Minimum Degree of Shear Connection (β_i) at PCC's (Clause 6.5, AS 2327.1)

Example 7.7-1 Calculation of β_i at PCC's where $\gamma \le 0.5$

The design bending moments and shear forces at three PCC's of a composite beam are given below. The details of the beam cross-section are given in Fig. 7.10. Determine the minimum degree of shear connection (β_i) required at the PCC's.

	M^* (kNm)	V^* (kN)	
PCC 1	220	210	
PCC 2	430	150	
PCC 3	558	0.0	Maximum moment cross-section

Solution

Using Table A4 of the Composite Beam Design Handbook [2],

$$\phi V_u = 547 \text{ kN}$$

At PCC 1, where the shear force is maximum,

$$\gamma = \frac{V^*}{\phi V_u} = 210/547 = 0.38 < 0.5$$

Since $\gamma < 0.5$ at PCC 1, moment-shear interaction is not relevant at any of the three PCC's. Therefore, the minimum degree of shear connection (β_i) can be determined using Eq. 6.5.2(1) or 6.5.2(2), as appropriate.

From Table A4, Composite Beam Design Handbook [2],

$$\phi M_s = 323 \text{ kNm}$$
$$\phi M_{b.5} = 516 \text{ kNm}$$
$$\phi M_{bc} = 590 \text{ kNm}$$

At PCC 1:

$$M^* = 220 \text{ kNm} < \phi M_s$$

Therefore, $\beta_1 = \underline{\mathbf{0.0}}$

At PCC 2:

Since $\phi M_s < M^* < \phi M_{b.5}$, using Eq. 6.5.2(1),

$$\beta_2 = \frac{M^* - \phi M_s}{2(\phi M_{b.5} - \phi M_s)}$$

$$= \frac{430 - 323}{2(516 - 323)}$$

$$\beta_2 = \underline{\mathbf{0.28}}$$

PCC 3 (Maximum moment cross-section):

Since $\phi M_{b.5} < M^* < \phi M_{bc}$, using Eq. 6.5.2(2),

$$\beta_3 = \frac{M^* + \phi M_{bc} - 2\phi M_{b.5}}{2(\phi M_{bc} - \phi M_{b.5})}$$

$$= \frac{558 + 590 - 2 \times 516}{2(590 - 516)}$$

$$\beta_3 = \underline{\mathbf{0.78}}$$

Since $\beta_3 > 0.5$, the condition given in Clause 6.6.2(a) of AS 2327.1 for the minimum value of β at the maximum moment cross-section is satisfied.

Example 7.7-2 Calculation of β_i at PCC's where $\gamma \geq 0.5$

The design bending moment and shear force at a PCC of a composite beam are 495 kNm and 365 kN, respectively. The details of the beam cross-section are given in Fig. 7.10. Determine the minimum degree of shear connection (β_i) required at the PCC.

Solution

As shown in Example 7.7-1,

$$\phi V_u = 547 \text{ kN}$$

Therefore,

$$\gamma = \frac{V^*}{\phi V_u}$$

$$= 365/547 = 0.67 > 0.5$$

Since $\gamma > 0.5$, moment-shear interaction occurs at the PCC. Therefore, the minimum degree of shear connection (β_i) can be determined using Eq. 6.5.3(1) or 6.5.3(2), as appropriate.

From Table A4, Composite Beam Design Handbook [2],

ϕM_s	=	323 kNm	ϕM_{bc}	=	590 kNm	ϕM_{bfc}	=	372 kNm
$\phi M_{b.\psi}$	=	541 kNm	ϕM_{sf}	=	242 kNm	ψ	=	0.64

Assuming $\beta_i > \psi$ and using Eq. 6.5.3(2),

$$\beta_i = \psi + \frac{(1-\psi)[M^* - 2(1-\gamma)\phi M_{b.\psi} - (2\gamma - 1)\phi M_{bfc}]}{2(1-\gamma)(\phi M_{bc} - \phi M_{b.\psi})}$$

$$\beta_i = 0.64 + \frac{(1-0.64)[495 - 2(1-0.67)541 - (2 \times 0.67 - 1)372]}{2(1-0.67)(590-541)}$$

$$\beta_i = \underline{\mathbf{0.77}} > \psi$$

The assumption of $\beta_i > \psi$ is correct.

7.8 Calculation of Concrete Compressive Force ($F_{cp.i}$) Corresponding to Minimum Degree of Shear Connection (β_i) (Clause 6.5, AS 2327.1)

Example 7.8-1 Calculate the values of $F_{cp.i}$ at PCC's in a Prismatic Composite Beam

The values of β_i calculated at the PCC's of a prismatic composite beam are given below (see Example 7.7-1). The cross-section of the beam is shown in Fig. 7.10. Calculate the concrete compressive force ($F_{cp.i}$) at each PCC.

	β_i
PCC 1	0.0
PCC 2	0.28
PCC 3	0.78

Solution

Using Table A4 of the Composite Beam Design Handbook [2], for the cross-section shown in Fig. 7.10,

$$F_{cc} = 2120 \text{ kN}$$

Concrete Compressive Force ($F_{cp.i}$) $= \beta_i F_{cc}$

At PCC 1:

$$\beta_1 = 0.0$$

Therefore, the steel beam alone is able to resist the design moment (i.e. $M^* > \phi M_s$).

$$\therefore \quad F_{cp.1} = \underline{\mathbf{0.0\ kN}}$$

At PCC 2:

$$\beta_2 = 0.28$$

$$F_{cp.2} = \beta_2 F_{cc}$$

$$= 0.28 \times 2120$$

$$F_{cp.2} = \underline{\mathbf{594\ kN}}$$

Noting that $f'_c = 32\,\text{MPa}$, the minimum number of shear connectors (n_2) required at PCC 2 can be determined using Table B1 of Composite Beam Design Handbook [2].

$$n_2 = \underline{7}$$

At PCC 3:

$$\beta_3 = 0.78$$

$$F_{cp.3} = \beta_3 F_{cc}$$

$$= 0.78 \times 2120$$

$$F_{cp.3} = \underline{\mathbf{1654\ kN}}$$

Noting that $f'_c = 32\,\text{MPa}$ and using Table B1 of Composite Beam Design Handbook [2],

$$n_3 = \underline{19}$$

Example 7.8-2 Calculate the Values of $F_{cp.i}$ at PCC's in a Non-prismatic Composite Beam

The details of a non-prismatic composite beam and the values of β_i calculated at PCC's are shown in Fig. 7.13. Calculate the concrete compressive force at each PCC.

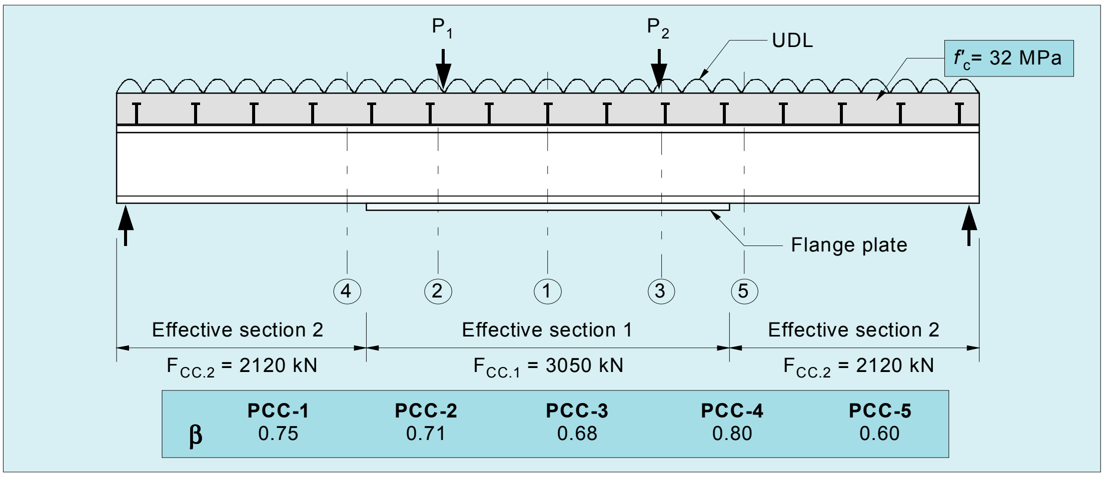


Figure 7.13 Details of Beam - Example 7.8-2

Solution

The beam has two effective sections, as shown in Fig. 7.13. The values of β_i for PCC's 1, 2 and 3 are calculated using $F_{cc.1}$.

$$F_{cc.1} = 3050 \text{ kN}$$
$$F_{cp.i} = \beta_i F_{cc.1}$$

Therefore,

$$F_{cc.1} = 0.75 \times 3050 = \underline{\mathbf{2288\ kN}}$$
$$F_{cp.2} = 0.71 \times 3050 = \underline{\mathbf{2166\ kN}}$$
$$F_{cp.3} = 0.68 \times 3050 = \underline{\mathbf{2074\ kN}}$$

Noting that $f_c^{'} = 32\,\text{MPa}$ and using Table B1 of Composite Beam Design Handbook [2], the minimum number of 19 mm studs required at PCC's are:

$$n_1 = \underline{\mathbf{26}}$$
$$n_2 = \underline{\mathbf{24}}$$
$$n_3 = \underline{\mathbf{23}}$$

The values of β_i for PCC's 4 and 5 are calculated using $F_{cc.2}$.

$$F_{cc.2} = 2120 \text{ kN}$$
$$F_{cp.i} = \beta_i F_{cc.2}$$

Therefore,

$$F_{cp.4} = 0.80 \times 2120 = \underline{\mathbf{1696\ kN}}$$
$$F_{cp.5} = 0.60 \times 2120 = \underline{\mathbf{1272\ kN}}$$

Using Table B1 of Composite Beam Design Handbook [2], the minimum number of 19 mm studs required at PCC's are:

$$n_4 = \underline{\mathbf{19}}$$
$$n_5 = \underline{\mathbf{15}}$$

8. REFERENCES

1. Standards Australia, *AS 2327.1-1996, Composite Structures, Part 1: Simply-Supported Beams*.
2. Australian Institute of Steel Construction and Standards Australia, *Composite Beam Design Handbook (SAA HB−1997)*, 1997.
3. British Standards Institution. *Eurocode 4: Design of composite steel and concrete structures, Part 1.1. General rules and rules for buildings (together with United Kingdom National Application Document)*, DD ENV 1994-1-1: 1994. London, 1994.
4. British Standards Institution, *BS 5950: Part 3: Section 3.1: 1990 Code of Practice for Design of Simple and Continuous Composite Beams*. London, 1990.
5. Patrick, M., Dayawansa, P.H., Eadie, I., Watson, K.B. and van der Kreek, N., *Australian Composite Structures Standard AS 2327, Part 1: Simply-Supported Beams*, Journal of the Australian Institute of Steel Construction, Vol. 29, No. 4, Dec. 1995, pp. 2-40.
6. Australian Institute of Steel Construction, *Design Capacity Tables for Structural Steel, Volume 1: Open Sections*, Second Edition, 1994.
7. OneSteel Reinforcing, *DECKMESH™*, September, 2000.
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APPENDIX A

DESIGN TABLES

A1 General

The data from Design Table A4 given in Composite Beam Design Handbook [2] were used in some of the worked examples given in Section 7. This design table is reproduced in Section A3 of this appendix for easy reference. The values are applicable to composite beams incorporating composite slabs with either BONDEK II or CONDECK HP profiled steel sheeting. Values similar to those in the table can readily be produced using the computer program COMPBEAM for different variable values, e.g. h_r = 58 mm for COMFORM.

The index to the design tables given in Composite Beam Design Handbook [2] is reproduced in Section A2 to indicate the ranges of the parameters covered by the design tables.

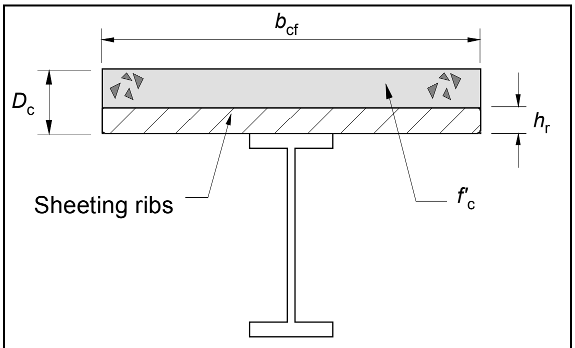
A2 Index to Design Tables in Composite Beam Design Handbook [2]

The index to the design tables is given below, and the notation is given in Figure A1.

Index to Design Tables

Table No.	b_{cf} (mm)	D_c (mm)	f'_c (MPa)	h_r (mm)	λ
WB's & UB's (300PLUS)					
A1	1200	120	25	55	1.0
A2	1200	120	25	55	0.0
A3	1200	120	32	55	1.0
A4	1200	120	32	55	0.0
A5	1600	120	25	55	1.0
A6	1600	120	25	55	0.0
A7	1600	120	32	55	1.0
A8	1600	120	32	55	0.0
A9	2100	120	25	55	1.0
A10	2100	120	25	55	0.0
A11	2100	120	32	55	1.0
A12	2100	120	32	55	0.0
UC's (300PLUS)					
A13	1200	120	25	55	1.0
A14	1200	120	25	55	0.0
A15	1200	120	32	55	1.0
A16	1200	120	32	55	0.0
A17	1600	120	25	55	1.0
A18	1600	120	25	55	0.0
A19	1600	120	32	55	1.0
A20	1600	120	32	55	0.0
A21	2100	120	25	55	1.0
A22	2100	120	25	55	0.0
A23	2100	120	32	55	1.0
A24	2100	120	32	55	0.0

Table No.	b_{cf} (mm)	D_c (mm)	f'_c (MPa)	h_r (mm)	λ
RHS's (G350)					
A25	1200	120	25	55	1.0
A26	1200	120	25	55	0.0
A27	1200	120	32	55	1.0
A28	1200	120	32	55	0.0
A29	1600	120	25	55	1.0
A30	1600	120	25	55	0.0
A31	1600	120	32	55	1.0
A32	1600	120	32	55	0.0
A33	2100	120	25	55	1.0
A34	2100	120	25	55	0.0
A35	2100	120	32	55	1.0
A36	2100	120	32	55	0.0



λ = 0, sheeting ribs perpendicular to beam
λ = 1.0, sheeting ribs parallel to beam

Figure A1 Notation

A3 Design Table A4 of Composite Beam Design Handbook [2]

TABLE A4: b_{cf} = 1200 mm, D_c = 120 mm, f'_c = 32 MPa, h_r = 55 mm, f_{yb} = 300 MPa
λ = 0.0 (i.e. sheeting ribs perpendicular to steel beam)

Steel section	ϕM_s	$\phi M_{b.5}$	ϕM_{bc}	ψ	$\phi M_{b.\psi}$	ϕM_{sf}	ϕM_{bfc}	ϕV_u	F_{cc}	19 mm studs for	I_{ti}	kD_b**	I_{tl}
	kNm	kNm	kNm		kNm	kNm	kNm	kN	kN	β =1.0*	10^6 mm^4	mm	10^6 mm^4
800WB192	2010	2410	2580	1.00	2580	1670	1880	1190	2120	24	4880	359	3770
800WB168	1700	2090	2260	1.00	2260	1360	1560	1190	2120	24	4270	341	3250
800WB146	1500	1890	2050	1.00	2050	1160	1350	1190	2120	24	3690	321	2760
800WB122	1180	1560	1720	1.00	1720	838	1020	1190	2120	24	3060	296	2250
700WB173	1640	2000	2160	1.00	2160	1330	1540	1100	2120	24	3610	320	2740
700WB150	1380	1740	1900	1.00	1900	1080	1280	1100	2120	24	3160	304	2360
700WB130	1220	1570	1730	1.00	1730	918	1110	1100	2120	24	2740	286	2020
700WB115	1030	1380	1540	1.00	1540	730	914	1100	2120	24	2400	270	1750
610UB125	934	1250	1400	1.00	1400	670	859	1180	2120	24	2040	256	1470
610UB113	834	1150	1280	1.00	1280	586	770	1100	2120	24	1870	244	1340
610UB101	785	1100	1230	0.95	1220	535	711	1100	2120	24	1690	233	1210
530UB 92.4	640	916	1020	0.92	1010	455	628	939	2120	24	1290	206	920
530UB 82.0	557	826	922	0.78	892	383	534	876	2120	24	1160	195	826
460UB 82.1	497	736	823	0.86	805	366	531	787	2120	24	932	179	660
460UB 74.6	449	679	761	0.78	731	329	481	719	2120	24	861	171	612
460UB 67.1	400	623	700	0.68	656	288	423	667	2120	24	787	162	562
410UB 59.7	323	516	590	0.64	541	242	372	547	2120	24	607	144	435
410UB 53.7	299	488	560	0.59	503	219	338	529	2120	24	551	136	397
360UB 56.7	273	448	522	0.63	471	209	337	496	2120	24	483	132	343
360UB 50.7	241	402	472	0.60	419	183	297	449	1960	22	437	124	314
360UB 44.7	214	366	432	0.58	379	154	263	420	1770	20	389	116	282
310UB 46.2	195	328	395	0.66	353	156	270	355	1780	20	330	111	238
310UB 40.4	178	299	363	0.66	322	142	248	320	1630	19	294	104	215
310UB 32.0	125	227	280	0.59	238	88.9	171	283	1220	14	231	90	173
250UB 37.3	138	243	303	0.68	267	112	213	283	1500	17	216	93	157
250UB 31.4	111	203	257	0.63	219	86.2	168	266	1250	15	183	85	136
250UB 25.7	87.9	162	207	0.63	175	68.6	134	214	1010	12	150	76	114
200UB 29.8	89.1	170	222	0.69	191	73.1	156	225	1200	14	137	78	101.0
200UB 25.4	72.3	142	187	0.65	157	57	125	203	1000	12	117	72	87.6
200UB 22.3	61.5	124	164	0.65	137	45.9	109	175	859	10	105	68	80.6
200UB 18.2	49.1	98.8	133	0.63	108	38.1	84.7	154	708	9	84.3	61	66.1
180UB 22.2	54.7	114	155	0.65	128	43.8	104	186	881	10	89.4	65	67.5
180UB 18.1	43.7	92.2	127	0.64	103	34.6	83.1	151	715	9	73.7	60	57.1
180UB 16.1	38.3	81.2	112	0.64	90.1	30.1	72.7	135	632	8	65.8	57	51.7
150UB 18.0	37.8	85.8	120	0.64	95.8	29.9	78.2	161	717	9	64.5	58	49.4
150UB 14.0	28.3	65.5	92.9	0.61	71.7	21.6	57.4	130	554	7	50.5	52	39.8

* Number of 19 mm headed-studs given in the table is that required for complete shear connection between a cross-section and each end of the beam.
** kD_b given in the table corresponds to I_{ti}.

APPENDIX B

NOTATION

The following notation is the same as that used in AS 2327.1–1996.

b	=	clear width of plate element outstand
b_1, b_2	=	centre-to-centre spacing of adjacent beams or distance from centre of steel beam to edge of slab outstand
b_{cf}	=	effective width of the concrete slab compression flange
b_{cr}	=	width of the concrete rib in a composite slab at mid-height of the steel ribs
	=	$s_r - b_{sr}$ (see Fig. 1.2.4 of AS 2327.1)
b_{e1}, b_{e2}	=	concrete slab effective width outstands on opposite sides of steel beam centre-line
b_f	=	width of steel beam flange
b_{sf1}	=	effective width of steel beam top flange
b_{sf2}	=	overall width of steel beam bottom flange
b_{sr}	=	width of steel rib in a composite slab at its mid height
D_c	=	overall depth of a concrete slab including the thickness of any profiled steel sheeting if present
D_s	=	overall depth of a steel beam
d_1	=	clear depth between flanges of a steel beam ignoring fillets or welds
d_h	=	calculated depth of the compressive zone below the top of the concrete slab at the strength limit state
F_{c1}	=	longitudinal compressive capacity of concrete cover slab within slab effective width
F_{c2}	=	longitudinal compressive capacity of concrete between steel sheeting ribs within slab effective width
F_{cc}	=	compressive force in concrete slab at a cross-section with complete shear connection where $\gamma \le 0.5$ at the strength limit state
F_{ccf}	=	compressive force in concrete slab at a cross-section with complete shear connection where $\gamma \le 1.0$ at the strength limit state
F_{cp}	=	compressive force in concrete slab at a cross-section with partial shear connection where $\gamma \le 0.5$ at the strength limit state
$F_{cp.i}$	=	value of F_{cp} corresponding to potentially critical cross-section i
F_{sc}	=	balancing compressive force in steel beam [either $(F_{st} - 2F_{cp})$ or $(F_{stf} - 2F_{ccf})$]
F_{st}	=	tensile capacity of steel beam, assuming that the entire cross-sectional area has yielded in tension
F_{stf}	=	tensile capacity of steel beam ignoring web(s), assuming that the entire cross-sectional area of the flanges has yielded in tension
f'_c	=	28-day characteristic compressive cylinder strength of concrete

f'_{cj}	=	estimated characteristic compressive strength of concrete at *j* days, but not taken as greater than f'_c
f_{ds}	=	design shear capacity of a shear connector
f_{vs}	=	nominal shear capacity of a shear connector
f_y	=	yield stress of steel used in design
f_{yf}, f_{yw}	=	yield stress of the flange and web, respectively, of the steel beam
G	=	total dead load
G_{sup}	=	superimposed dead load
h_r	=	height of the steel ribs in profiled steel sheeting
L_{ef}	=	effective span of a composite beam
M^*	=	design bending moment at a cross-section
M_b	=	nominal moment capacity of a composite cross-section where $\gamma \le 0.5$ and $0 < \beta < 1.0$
$M_{b.5}$	=	value of M_b corresponding to $\beta = 0.5$
$M_{b.\psi}$	=	value of M_b corresponding to $\beta = \psi$
M_{bc}	=	nominal moment capacity of a composite beam cross-section where $\gamma \le 0.5$ and $\beta = 1.0$
M_{bf}	=	nominal moment capacity of a composite beam cross-section where $\gamma = 1.0$ and $0 < \beta < \psi$, neglecting any contribution of the steel beam web(s)
M_{bfc}	=	nominal moment capacity of a composite beam cross-section where $\gamma = 1.0$ and $\beta = 1.0$, neglecting any contribution of the steel beam web(s)
M_{bv}	=	nominal moment capacity of a composite beam cross-section where $0 < \gamma < 1.0$ and $0 \le \beta \le 1.0$ (*Note: This is the general term used for nominal moment capacity.*)
M_s	=	nominal moment capacity of steel beam section
M_{sf}	=	nominal moment capacity of steel beam section neglecting any contribution of the web(s)
n	=	total number of shear connectors provided between a cross-section and an end of the composite beam
n_A	=	number of shear connectors between a potentially critical cross-section (i) and end A of beam
n_B	=	number of shear connectors between a potentially critical cross-section (i) and end B of beam
n_i	=	minimum number of shear connectors (with the same design shear capacity f_{ds}) between a potentially critical cross-section (i) and the ends of the beam to satisfy the design requirement $\phi M_{bv} \ge M^*$
n_c	=	minimum number of shear connectors (with the same design shear capacity f_{ds}) required between a potentially critical cross-section (*i*) and the ends of the beam to achieve complete shear connection
PNA	=	plastic neutral axis in the steel beam
Q	=	live load
s_r	=	transverse spacing of steel ribs in a profiled steel sheet (See Fig. 1.2.4 of AS 2327.1)

t_f	=	thickness of the flange of a steel beam
t_{f1}, t_{f2}	=	value of t_f corresponding to the top and bottom flanges, respectively
t_w	=	thickness of the web(s) of a steel beam
t'_w	=	effective thickness of non-compact web(s) of a steel beam
V^*	=	design vertical shear force acting at a composite beam cross-section
V_u	=	nominal vertical shear capacity of a composite beam cross-section at the strength limit state
x_e	=	distance from top of steel beam to elastic neutral axis
x_p	=	distance from top of steel beam to plastic neutral axis
β	=	degree of shear connection at a cross-section
	=	$\dfrac{F_{cp}}{F_{cc}}$
β_i	=	minimum degree of shear connection at a potentially critical cross-section *i* to satisfy the design requirement $\phi M_{bv} \geq M^*$
β_m	=	degree of shear connection at the maximum moment cross-section of a composite beam
γ	=	shear ratio at a composite beam cross-section
	=	$\dfrac{V^*}{\phi V_u} \leq 1.0$
θ	=	acute angle between the steel ribs of a composite slab and the longitudinal axis of the steel beam
λ	=	factor accounting for the inclination of profiled steel sheeting ribs with respect to longitudinal axis of steel beam
λ_e	=	plate element slenderness
λ_{ep}	=	plate element plasticity slenderness limit
λ_{ey}	=	plate element yield slenderness limit
ϕ	=	capacity factor relevant to a strength limit state
ψ	=	value of β corresponding to complete shear connection of a composite beam ignoring the presence of the steel beam web(s)
	=	$\dfrac{F_{ccf}}{F_{cc}}$