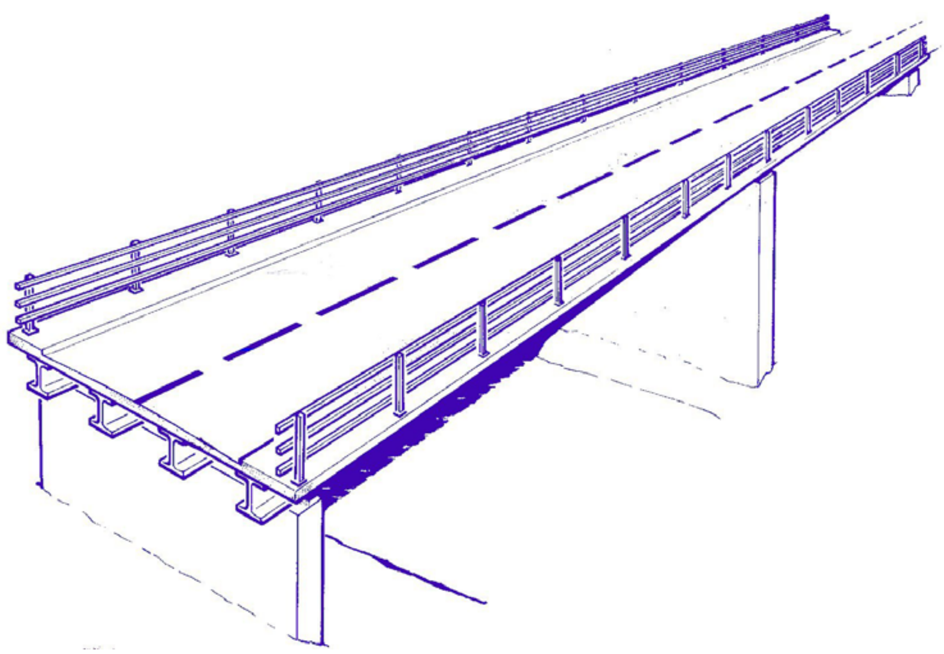


Bridge Design to the Eurocodes

Simplified rules for use in student projects

(Document RT1156)

Version	Date of Issue	Purpose	Author	Technical Reviewer	Approved
1		Distribution to universities			
2	4/2/11	Footway loading added			
3					

SCI reference: RT1156

The Steel Construction Institute
Silwood Park
Ascot
Berkshire, SL5 7QN.

Telephone: +44 (0) 1344 636525
Fax: +44 (0) 1344 636570
Email: reception@steel-sci.com

www.steel-sci.org

FOREWORD

This document is intended to offer student engineers an introduction to bridge design using the Structural Eurocodes. A simplified version of the comprehensive rules in the Parts of the Eurocodes that relate to composite (steel and concrete) bridge design has been derived and some general guidance and commentary has been included.

The rules and guidance relate principally to the structural steel elements of the bridge. It is assumed that sufficient background of stability of steel structures has already been covered in the regular lectures.

This simplified guidance complements the *Student guide to steel bridge design*, published by Corus (now Tata Steel), which is also available in pdf format on the Tata Steel and SCI web sites) and is particularly appropriate for use in undergraduate design projects in the annual SCI/Tata Steel/BCSA undergraduate design competition.

Bridge Design to the Eurocodes
Simplified rules for use in student projects

CONTENTS

Introduction to the simplified rules

(For use in student projects)

CONTENTS

1 GENERAL

The purpose of a bridge is to carry a service over an 'obstacle' (which may be another road or railway, a river, a valley etc). The designer has to ensure that bridge designs are both safe and economic.

Composite construction is a common and economic form of construction for ordinary highway bridges and general guidance on the design of such bridges is given in the Corus publication *Student guide to steel bridge design*. The present document complements that publication by presenting more detailed design guidance, based on the principles and rules set out in the structural Eurocodes.

This document presents a simplified version of selected sections of some of the relevant Eurocode Parts, in order to give an outline appreciation of the procedures necessary to verify the design of a composite bridge structure. It has been written to explain both the Eurocode provisions and the background structural concepts at easily understood levels. It is emphasised that a bridge designed to this simplified version will not necessarily meet all the requirements of the Eurocodes: some clauses that are important are not covered because of their complexity; the loading applied to the structure has been greatly simplified; only certain constructional configurations and details are covered.

2 THE STRUCTURAL EUROCODES

The Structural Eurocodes are a harmonized set of structural design standards, developed by CEN over the last 30 years, to cover the design of all types of structures in steel, concrete, timber, masonry and aluminium. There are ten Eurocodes, each published in a number of separate Parts. The ten Eurocodes are:

- Eurocode: Basis of structural design
- Eurocode 1: Actions on structures
- Eurocode 2: Design of concrete structures
- Eurocode 3: Design of steel structures
- Eurocode 4: Design of composite steel and concrete structures
- Eurocode 5: Design of timber structures
- Eurocode 6: Design of masonry structures
- Eurocode 7: Geotechnical design
- Eurocode 8: Design of structures for earthquake resistance
- Eurocode 9: Design of aluminium structures

The Eurocodes are designated EN 1990 to EN 1999 respectively.

Each of these 10 Eurocodes are in a number of separate Parts - there are 58 Parts in total. Some Parts give general rules, some Parts give rules applicable to one form of construction. Composite bridge design is covered by 'Part 2' of the three material Eurocodes EN 1992, EN 1993 and EN 1994, and traffic loading is covered by Part 2 of EN 1991. These Parts are designated EN 1992-2, EN 1993-2, EN 1994-2 and EN 1991-2 respectively.

Although the Eurocodes are harmonized documents that are applicable throughout Europe, certain provisions, such as the setting of partial factors for safety, are chosen by the national standards bodies (such as BSI). The Eurocodes will thus be accompanied by National Annexes that set out those national choices.

3 SIMPLIFIED EUROCODE RULES

This document presents simplified versions of selected sections of EN 1990, EN 1991-2 EN 1993-2 and EN 1994-2. The selection is intended to provide rules for use in student undergraduate composite highway bridge design projects.

The rules provided in this document are intended to cover the principal design verifications needed for a composite highway bridge. The rules can be applied to single or continuous spans of either multi-girder or ladder deck construction (see the Corus publication for illustrations of these forms).

Part A explains the design basis. It sets out the basic requirements and explains how limit state design methods are applied to composite bridges.

Part B sets out the loading to be considered. Primarily this relates to traffic loading but mention is also made of the temperature variation. No information is given about wind loading - in most cases wind will have no significant effect on this type and size of bridge; determination of wind loads on bridges where the loads would make a significant contribution is too complex for simplified rules.

Part C sets out rules for designing steel bridges. All-steel construction is rarely used within the range of bridges that is likely to be encountered in an undergraduate design project. However, during construction of composite bridges, before the concrete has cured, the steelwork carries all the loading and its ability to carry that weight has to be verified. Further, the design of the composite girders, for the in-service life of the bridge, relies partly on the detailed evaluation of the performance of the steel elements and numerous references are made to the steel design rules. Rules for the design of steel elements are therefore an essential part of the design of a composite bridge.

Part D sets out the rules for composite bridge design. The rules cover the design verification of the composite girders, making reference to Part C where necessary, and of the shear connection between steel and concrete that is essential for composite action.

Simplified rules for reinforced concrete bridges are not included in this document. Some aspects of RC design have been covered in Part D. Design of the deck slab would require the application of EN 1992-2 but the detailed design of deck slabs is outside the scope of the projects to which this document applies. (It has been assumed, in preparing this document, that the design brief will define the details of the deck slab and its reinforcement, rather than demanding the detailed design of that element.)

4 FURTHER INFORMATION

If a more detailed understanding of the Eurocode rules is needed, reference should be made to the Standards themselves, as published by BSI.

Guidance on the use of Eurocodes for the design of real composite highway bridges is due to be published by SCI in late 2007/early 2008. The following publications will be issued.

- Composite highway bridges: Design to the Eurocodes (P356)
- Composite highway bridges: Worked Examples (P357)

For a full background to the bridge Eurocodes, see the Guides published by Thomas Telford. However, these are comprehensive and detailed documents that are generally beyond the scope that would normally need to be considered by undergraduate students.

Visit www.eurocodes.co.uk or www.thomastelford.com/books.

Part A - Basis of structural design
(Simplified version of EN1990)

(For use in student projects)

CONTENTS

1 GENERAL

This part of the document presents a simplified version of the basis of structural design for bridges for the use in undergraduate design projects. It is based generally on EN 1990 and is a much-reduced version of the comprehensive statements that are included in the Eurocode document.

1.1 Basic requirements

The basic requirements are simply that a structure should be built such that, over its intended life, it will sustain the actions and influences upon it and remain fit for use. This means that it must have a certain strength, perform in an acceptable manner and be durable.

To provide design methods to achieve these requirements, a reliability approach is adopted. Design values are determined such that they have a known statistical probability of being achieved - the values of actions (loads) have a known (low) probability of not being exceeded, the values for strength have a known (high) probability level of being achieved. The design procedure is then to model and evaluate the behaviour of a structural model in order to verify that calculated effects due to the actions do not exceed the design strength/ deformation limits. The reliability approach is achieved through the use of limit state design principles.

1.2 Limit states design

EN 1990 sets out limiting design situations at which the design values of the effects of imposed loading or imposed displacement just reach the design values of the strength or the acceptable deformation of the structural components.

Two basic limit states are defined - ultimate limit state and serviceability limit state.

Ultimate limit states

Collapse, failure, overturning, buckling or rupture of the whole or of part of the structure are classed as ultimate limit states (ULS). Fatigue failure is also an ultimate limit state.

Serviceability limit states

Irreversible deformation that would affect the performance of the structure, deformation that affects the appearance of the structure and uncomfortable dynamic performance are all serviceability limit states (SLS). Irreversible deformation relates mainly to the yielding of steelwork (other than local redistribution of high-spot stresses due to the fabrication process); dynamic performance relates to natural frequencies of vibration and the accelerations experience by users.

1.3 Partial factors method

In the verification using the partial factors method, design values are determined from prescribed values and rules set out in the various Parts of the Eurocodes and on ‘partial factors on safety’ to adjust those values according to the design situation.

Partial factors are applied to actions in order to determine design load effects (loads); other partial factors are applied to calculated resistances (determined from material strengths and geometric properties) to determine the design value of the resistance. The values of the factors are set to achieve an overall level of reliability for a given return period (the design life of the structure).

For use in these simplified procedures, all the relevant prescribed values and partial factors are given in this and the other parts of the document.

2 DESIGN VARIABLES

Symbols and subscripts

Design variables are designated by a wide range of symbols and subscripts, according to an agreed convention. Commonly used symbols and subscripts are defined at the start of the other sections in this document.

Actions

An action is a set of forces (loads) or imposed deformations (e.g. due to temperature change). The prescribed values, referred to as characteristic values and denoted by a symbol with the subscript 'k' are given in Part B.

Effects of actions

The effect of an action is an internal moment, axial force, stress, strain or deflection of the structure. The effects of characteristic values of actions also have the subscript 'k'.

Material properties

Material properties, such as the yield strength of steel, are also specified in terms of characteristic values but often such characteristic values are in fact specified minimum values according to a product Standard and although strictly the 'k' subscript should be used, it is often omitted.

Resistance

The resistance of a structural member or component is its capacity to withstand the effects of actions - typically it is the axial resistance, bending resistance or shear resistance of a member. Resistance is determined from material properties and member geometry and is usually expressed as design resistance (which means that the characteristic value has been divided by a partial factor) and denoted by the subscript 'Rd'.

3 VERIFICATION AT ULTIMATE LIMIT STATES

At ULS it must be verified that:

$$E_{\mathrm{d}} \leq R_{\mathrm{d}}$$

Where E_{d} is the design value of the effects of actions (internal moment, axial force, etc.) and R_{d} is the design value of the corresponding resistance

3.1 Effects of actions

At ULS, the effects of the actions (i.e. the internal bending moments, axial forces etc. due to the applied loadings and displacements) are expressed in terms of combinations of actions that can occur simultaneously. The basic expression is:

$$E\left(\sum \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}\right) \tag{3.1}$$

Where $G_{k,j}$ is the characteristic value of the j-th permanent action, P is the permanent action caused by controlled forces or deformation (prestressing), $Q_{k,1}$ is the characteristic value of the 'leading' variable action and $Q_{k,i}$ are the accompanying variable actions. The $E(\ \ )$ denotes 'the effect of' and the '+' signs denote the combination of effects due to the separate actions.

Permanent actions are self weight - typically the weight of steel, concrete and superimposed load such as surfacing and parapets; the partial factors γ_G applied to each type of permanent action may be different, hence the summation term and the j index subscript.

Apart from the familiar 'prestressing' that is used for some reinforced concrete structures (internal tendons, bars, etc.), which is outside the scope of this document, prestressing actions also include forces in structural components such as cable stays, because these are usually manufactured to an unstressed length shorter than the nominal length in the completed structure - see further comment in Section 15 of Part C. The γ_P factor is defined in the relevant Eurocode Part. In most undergraduate design projects, prestressing actions may be ignored.

The variable actions are either direct (the weight of traffic, the wind pressure, etc.) or indirect (expansion/contraction due to temperature). The partial factors γ_Q depend on the type of action and its predictability. It is unlikely that the most adverse loading from one action will occur simultaneously with that due to a different action. In recognition of this, EN 1990 refers to one action as a 'leading action' and the other actions as 'accompanying; a reduction factor ψ is applied to accompanying actions. In principle, each different action should in turn be considered as the leading action, to determine which combination of leading and accompanying actions is the most onerous, but for simplified highway bridge design it may be assumed that the traffic loading is the leading action. See further discussion in Section 2.3 of Part B of this document about temperature and wind loading.

There are similar expressions for combinations of actions in accidental and seismic situations, each with a different set of partial factors, but these are not of concern for simplified design.

3.2 Design value of resistance

The design value of resistance is given by the relevant material Eurocode and its value is determined from characteristic resistance values divided by partial factors on material strength γ_M. The values of γ_M are given in Parts C and D of this document

4 VERIFICATION AT SERVICEABILITY LIMIT STATES

At SLS it must be verified that:

$$E_{\mathrm{d}} \leq C_{\mathrm{d}}$$

Where E_{d} is the design value of the effects of actions in the SLS criterion and C_{d} is the limiting design value of the relevant criterion.

4.1 Effects of actions

At SLS there are in principle three combinations of actions to consider: characteristic, frequent and quasi-permanent. For bridges the characteristic combination is used for checking that no inelastic response occurs; the frequent combination is used if deflection needs to be checked (this includes evaluation of dynamic response). The quasi-permanent combination relates to long-term effects; for bridges, provided that the appropriate modulus of elasticity is used for long-term actions, this combination only needs to be considered when determining crack widths in concrete. Only the characteristic combination is relevant to simplified design; the other combinations are not discussed further here.

For the characteristic loading combination, the same characteristic values of actions are used as at ULS but all the γ factors are taken as unity. Thus the expression becomes:

$$E\left(\sum G_{\mathrm{k},j} + P + Q_{\mathrm{k},1} + \sum_{i>1} \psi_{0,i} Q_{\mathrm{k},i}\right) \tag{4.1}$$

4.2 Serviceability criteria

The SLS criterion that might need to be considered in simplified design is the limitation that stresses in steel should not exceed the yield stress. This limitation would need to be considered if the ULS design resistance were based on plastic bending resistance - it must then be verified that the stress calculated elastically at SLS does not exceed yield.

No partial factor is applied to yield stress (strictly, $\gamma_{\mathrm{M}} = 1$).

The limitation on crack widths in concrete in tension, for durability reasons, is mentioned in Section 9 of Part D but crack widths do not need to be checked in simplified project designs.

There are also serviceability limitations in relation to deflection (for example, ensuring that the deflected shape does not infringe a clearance gauge under the frequent loading combination) and vibration performance (for example to ensure that users do not experience discomfort) but these are unlikely to be of concern in simplified project designs and no guidance on these limitations is given here.

Part B - Loading on bridges
(Simplified version of EN 1991)

(For use in student projects)

CONTENTS

1 GENERAL

1.1 Scope of this Part

This Part presents the rules for determining the actions to be applied to a highway bridge. It covers permanent actions and variable actions due to the weight of traffic. Actions due to temperature variation and wind are mentioned, although they will not usually need to be considered in student project designs.

The values of the actions and the factors to be applied to the actions are generally based on those in the UK National Annexes to EN 1991-2 and EN 1990-A2. Those values differ from the values in the harmonized versions of the Eurocodes

Fatigue loading is not covered - although composite bridges usually require fatigue assessment, fatigue design is outside the scope of this document.

Accidental actions are also not covered.

1.2 Symbols

Some of the main variables used in the design of steelwork are:

G	Permanent action (e.g. self weight)
Q	Variable action (e.g. traffic loads)
A	Accidental action (e.g. impact from vehicles)
γ	Partial factor (applied to action)
ψ	Factor applied to accompanying action
g	Permanent action/unit area
q	Variable action/unit area

The two main subscripts used for actions are k, for characteristic values, and d, for design values. These are often used in conjunction with indices (i or j) when there are several actions of one particular type.

1.3 Partial factors on actions

The values to be used are given in Table 1.1

Table 1.1 *Partial factors on actions*

Partial factor on:		ULS	SLS
Permanent actions (concrete weight)	γ_G	1.35	1.00
Permanent actions (steel weight and super-imposed loads)	γ_G	1.20	1.00
Variable actions (road traffic)	γ_Q	1.35	1.00
Wind actions	γ_Q	1.70	1.00
Thermal actions	γ_Q	1.50	1.00

1.4 Factors on accompanying actions

For most student project designs, only the leading action (traffic loads) will need to be considered and thus factors for accompanying actions will not be needed.

However, if the project brief provides details of wind loads or thermal actions, the combination of leading and accompanying actions will need to be considered. Table 1.2 presents the values of the ψ_0 factors for accompanying actions.

Table 1.2 *Factors on accompanying actions*

Factor on:	ψ_0
Normal traffic loads (Load Model 1)	0.75
Special vehicles (Load Model 3)	0
Wind forces	0.8
Thermal actions	0

Thus, if traffic load is the leading action, only 80% of the wind forces need to be considered as accompanying actions. If wind forces or thermal actions are considered as the leading action, only 75% of normal traffic loads and no special vehicles need be considered as accompanying actions.

2 ACTIONS

2.1 Permanent actions

Self weight of structural elements

The self-weight of the structural components of the bridge may be based on the following specific weights:

Steel 77 kN/m^3

Concrete 24 kN/m^3

Superimposed dead loads

The weight of a 100 mm thick layer of surfacing may be taken as 2.2 kN/m^2.

The weight of a parapet may be taken as 0.5 kN/m.

2.2 Traffic loads

Several traffic load models are given in EN 1991-2, the most important of which is Load Model 1, representing normal traffic. This is the only traffic load model that need be considered in most student project designs. In practice, Load Model 3, representing abnormal vehicles, also has to be considered. The values for both load models are specified in the UK National Annex to EN 1991-2. The values given below are based on that NA.

2.2.1 Load Model 1 (LM1)

Load Model 1 comprises two components: a uniformly distributed load over the full width of a traffic lane and a pair of axles.

For the UDL, the length of lane loaded should be those parts of the influence line that lead to adverse load effects. (Thus, in a continuous three-span bridge, the central span should be loaded for sagging moments at midspan and two adjacent spans should be loaded for hogging moment at the intermediate support between the spans.)

In each lane (up to a maximum of three lanes) a pair of axles (referred to as a Tandem System, TS) should be positioned centrally in the lane at the position along the lane that causes maximum adverse effect. (Thus, for the three span bridge mentioned above, the TS would be at midspan for sagging moments and part way into the central span for hogging moments.)

The characteristic values of the two components of LM1 are:

UDL q_k = 5.5 kN/m^2

TS Q_k = 300 kN (on each axle, i.e. 150 kN per wheel))

The arrangement of LM1 is shown in Figure 2.1.

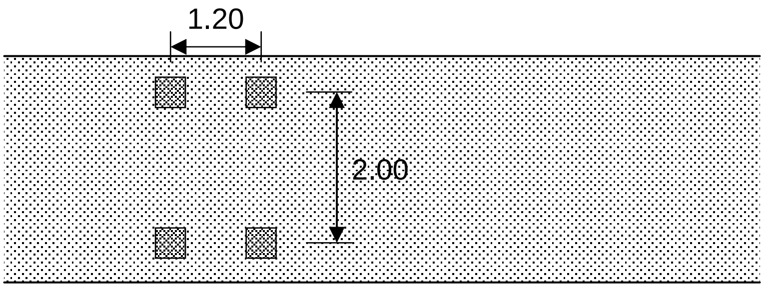


Figure 2.1 *Load Model 1- arrangement of Tandem System within loaded lane*

2.2.2 Load Model 3

Load Model 3 will not normally be required to be considered in student design projects but for information, one of the special vehicle models in the UK NA is shown in Figure 2.2. The value of 165 kN is the load per axle.

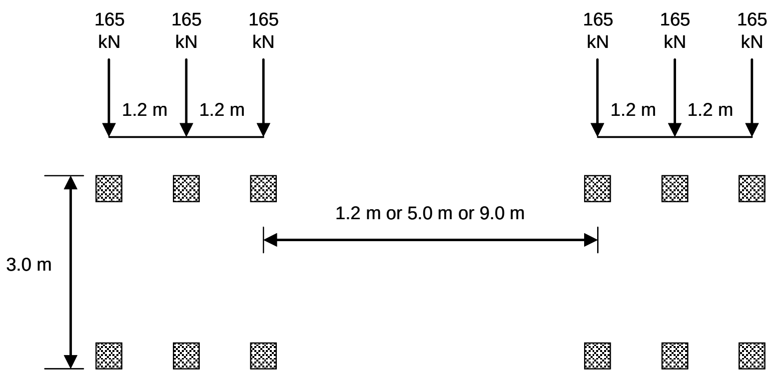


Figure 2.2 *Load Model 3 vehicle*

The LM3 vehicle replaces the LM1 UDL in one lane over a length from 5 m ahead of the first LM3 wheel to 5 m behind the last wheel. The LM1 TS load still has to be applied, elsewhere in that lane.

2.2.3 Footway loading

For footbridges and combined footway/cycle track bridges, a characteristic UDL of 5 kN/m^2 should be applied on all adverse areas of the influence surface, unless otherwise specified in the project brief.

For footways on highway bridges, a characteristic UDL of 3 kN/^{2} should be applied on the footway on all adverse areas, unless otherwise specified in the project brief. This load should be applied in combination with LM1 and the combined loading treated as a single variable action when considering combinations of actions.

2.3 Thermal actions and wind loads

Thermal actions and wind loads will not normally need to be considered in student project designs but in case the particular brief defines such actions, the following should be noted.

Thermal action is essentially the expansion and contraction due to temperature change. The values of expansion/contraction may be calculated using the thermal coefficient given in Section 3.2 of Part C (use this for both steel and concrete). The consequences of the thermal action might include the introduction of eccentricity of bearing reaction relative to bearing support stiffeners above sliding bearings; on a long viaduct, the size of the movement joint at the end of the bridge will need to be determined.

EN 1991-1-5 also specifies values of differential temperature - the variation of temperature vertically through the bridge cross section. This thermal action would produce a vertical curvature over a simply supported span but in a continuous bridge the vertical curvature is restricted and consequently secondary bending moments are introduced. These effects need not be considered in simplified designs.

EN 1991 provides detailed rules for determining wind forces, depending on environment factors and the shape of the bridge superstructure. If wind forces are to be considered in a student project, the wind pressure will be specified in the project brief.

3 TRAFFIC LANES

For the application of the traffic loading, the positions of the traffic lanes within the carriageway must be defined.

The carriageway is the running surface of the road, including all traffic lanes, hard shoulders, and white line markings. Where there are raised kerbs, it is the width between kerbs. In the absence of kerbs, it is the width between safety barriers.

For application of the traffic models, 3 m wide notional lanes should be located on the carriageway in such a way that the UDL will cause maximum adverse effect. Any width of lane (less than 3 m wide) is referred to as a "remaining area". The effect of this is shown in Figure 3.2.

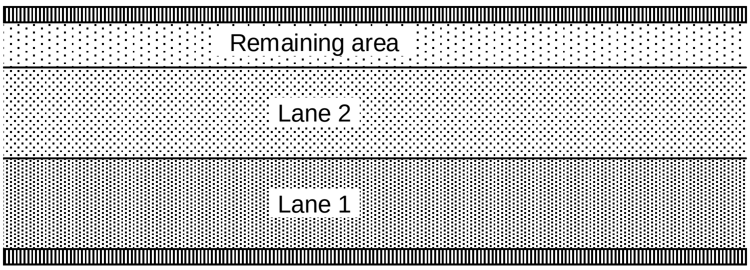


Figure 3.2 *Notional traffic lanes within a carriageway*

Since the value of the UDL is uniform in all lanes (according to the UK NA), it can simply be applied to the full width of the carriageway.

RT 1156 version 02, February 2011

Also, it is probably much easier to apply a knife edge load (KEL) for each axle (i.e. distribute the weight of one axle along a line across the lane) than two isolated wheel loads, and this is adequate for global analysis. As a further simplification, where the TS are intended to be side by side, the total weight of all the TS axles may be applied as a single KEL across the width of the carriageway - the reduction in action effects (relative to accurately positioned wheel loads) is very modest for global analysis.

Part C - Design of steel bridges
(Simplified version of EN 1993-2)

(For use in student projects)

CONTENTS

1 GENERAL

1.1 Scope of this Part

This Part presents rules for verifying the adequacy of steel beams in bridges. The rules in this Part only cover the verification of beam elements and their connections, restraints and attachments. The design of trusses, box girders and plate girders with longitudinal stiffeners are outside the scope of this document.

The simplified procedures for the design of steelwork components, assemblies and connections are based on rules in EN 1993-2. That Part of the Eurocode refers to general rules in EN 1993-1-1 and to various other Parts, such as EN 1993-1-8 (for connection design). Where appropriate, procedures based on those other Parts have also been included, although the simplified nature of this document has meant that only a few such inclusions have been made.

Fatigue design

If a stress or range of stress is applied repetitively to an element of a structure, it may fail prematurely by fatigue at a stress below (sometimes well below) its static strength. The loading to be considered in fatigue design is set out in EN 1991-2 and the rules for calculating fatigue resistance and verifying its adequacy are given in EN 1993-1-9. In practice, the fatigue assessment of highway bridges does not usually govern the sizes and configuration of main members but design for adequate fatigue life often affects the connection details (welding and bolting details). Fatigue design is outside the scope of this simplified document.

Avoidance of brittle fracture

To avoid brittle fracture at low temperature, steel material needs to have sufficient 'notch toughness'. This is achieved by specifying a suitable grade of steel: suitable grades for most steel bridges are available in BS EN 10025. Detailed procedures for the selection of the appropriate grade is also outside the scope of this document but a simple guidance is given in the Corus publication *Student guide to steel bridge design*.

1.2 Symbols

Some of the main variables used in the design of steelwork are:

M	Bending moment	A	Area
N	Axial force	I	Second moment of area
V	Shear force	W	Section modulus
T	Torsional moment	$\bar{\lambda}$	Normalized slenderness
F	Force	χ	Reduction factor (for buckling)

Some of the main subscripts are

Ed	design effect	b	buckling
Rd	design resistance	cr	critical
Rk	characteristic resistance	op	out of plane
el	elastic	c	related to cross section
pl	plastic	y, z	related to y-y or z-z axis

Thus:

N_{Ed} is the design value of axial force (an effect)

$N_{c,Rd}$ is the design resistance of the cross section

$N_{b,Rd}$ is the design buckling resistance (of a member)

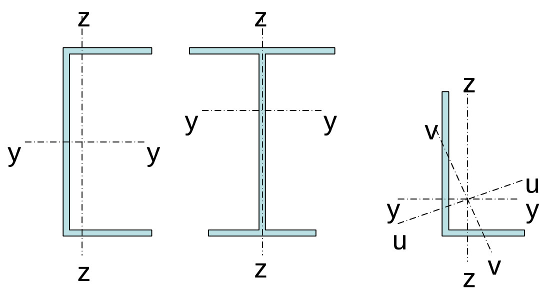
M_{cr} is the elastic critical moment (due to lateral torsion buckling of a beam)

Generally, the rigorous use of these subscripts does contribute significantly to clarity in calculations: it should be clear from the subscripts whether the variable is an effect or a resistance, gross or effective, elastic or plastic, related to a plate of a stiffener, etc. although this does sometimes result in lengthy subscripts.

1.3 Geometrical axes

The sign convention for axes differs from traditional UK convention but is compatible with many software analysis packages.

x – along the member



2 BASIS OF DESIGN

2.1 Ultimate limit state (ULS)

All beams should be designed to provide adequate strength to resist the design effects of the actions, using partial factors that are appropriate to the ULS. The following need to be considered.

- Material strength.
- Limitations on shape on account of local buckling of individual elements (i.e. webs and flanges).
- Moment resistance of cross sections.
- Effective sections (reductions for compression buckling and holes).
- Lateral torsional buckling.
- Web buckling (governed by depth to thickness ratio of web and panel size).
- Combined bending and shear effects.

(Resistance to coexistent axial forces is not considered in this simplified guidance, apart from a brief mention in Section 15. Axial forces are unlikely to be significant for beams in student projects.)

2.2 Serviceability limit state (SLS)

Beams should also be designed to ensure that no yielding or permanent deformation takes place under the (lower) design effects that are appropriate to SLS.

When the beam has been designed at the ULS as a class 3 or class 4 section (see Section 5.1) the resistance is based on an essentially elastic behaviour; the requirements at the SLS are automatically satisfied and no further checks need be made.

When the beam has been designed at the ULS as a class 1 or class 2 section, (see Section 5.1) utilising the plastic moment capacity, it is quite possible that yielding could occur in extreme fibres under the SLS characteristic loading. Beams of class 1 or 2 section must therefore be checked at the SLS, but in that case a linear elastic stress distribution must be used, i.e. the beams must be treated in the same manner as class 3 beams.

2.3 Partial factors

The values of γ_M to be used are given in Table 2.1

Table 2.1 *Partial factors γ_M*

Partial factor on:		ULS	SLS
Resistance of cross sections	γ_{M0}	1.00	
Resistance of members to instability	γ_{M1}	1.10	
Resistance of joints	γ_{M2}	1.25	
Slip resistance (Category C joints)	γ_{M3}	1.25	
Slip resistance (Category B joints)	$\gamma_{M3,ser}$		1.10
Nominal stresses due to SLS characteristic loads	$\gamma_{M,ser}$		1.00

2.3.2 Structural Support

Provisions should be made in the design for the transmission of vertical, longitudinal and lateral forces to the bearing and supporting structures.

3 PROPERTIES OF MATERIALS

3.1 Yield strength

The yield strength of steel manufactured in accordance with BS EN 10025 should be taken as:

Grade S275 $f_y = 275$ N/mm^2
Grade S355 $f_y = 355$ N/mm^2

In most cases, grade S355 should be used, for economy.

3.2 Mechanical properties of steel

Modulus of elasticity, $E = 210\,000$ N/mm^2
Shear Modulus $G = 80\,000$ N/mm^2
Poisson's ratio $\nu = 0.3$
Coefficient of thermal expansion $\alpha = 12 \times 10^{-6}/^{\circ}$C

4 GLOBAL ANALYSIS FOR LOAD EFFECTS

4.1 General

Elastic global analysis should be used to determine the load effects (internal forces and bending moments). Plastic analysis of the structure (i.e. redistribution of moments due to plastic hinge formation) should not be used.

4.2 Section properties

Gross section properties should be used in global analysis. (In girders of the proportions likely to be considered in undergraduate projects, there is no significant shear lag.)

5 CLASSIFICATION OF CROSS SECTIONS

5.1 General

The capacity of a section can be limited by local buckling of the flange or web in compression.

Local buckling is mainly of concern in thin-walled sections and occurs when the elements of the member are sufficiently slender for short wavelength compression buckles to develop before yield stress is reached in the element.

Such effects are guarded against by limiting the proportions of the elements (outstands of compression flanges and depths of webs). Four classes of section are distinguished:

(a) Class 1, where the full plastic moment of the cross section can be developed and there is sufficient rotation capacity to form a plastic hinge (although plastic global analysis is not allowed in bridge design - see Sections 4.1 and 4.3 of Part D - and thus hinges do not form).

(b) Class 2 where the full plastic moment of the cross section can be developed but there is insufficient rotation capacity to form a plastic hinge.

(c) Class 3 where the stress at the extreme fibre assuming a linear elastic distribution of stress can reach the yield strength but local buckling would prevent the development of the plastic resistance of the cross section.

(d) Class 4, where local buckling occurs before the attainment of yield stress at the extreme fibre and thus the resistance of the cross section is less than that if it were a class 3 section.

The classification of a section depends on the width to thickness ratio of elements of the cross-section; ratios presented in the following section are used to determine the classification.

For class 1 and class 2 sections the stress distribution for determining the bending resistance is given by a rectangular block, as shown in Figure 5.1. Since considerable plastic yielding is necessary to develop this distribution, the width to thickness limitations are more stringent than those for class 3 sections.

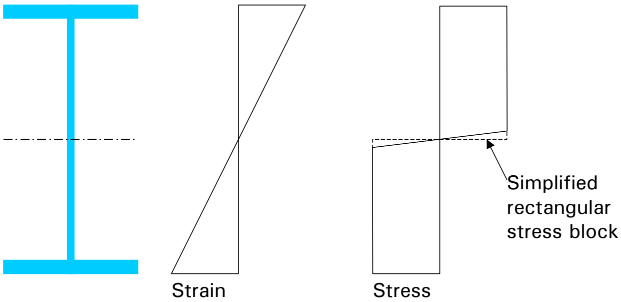
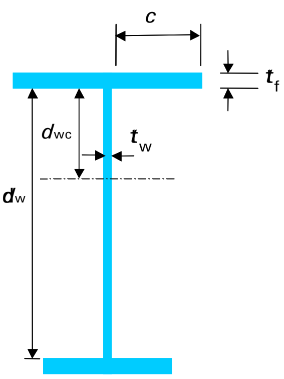


Figure 5.1 *Bending stresses and strains in a class 1 or class 2 cross section designed to plastic resistance*

5.2 Limiting width to thickness ratios

Limiting ratios for the flanges and webs of I sections are given below. The whole section is classified as the lower of the classes of the flange and the web. Relevant dimensions are shown in Figure 5.2. Cross sections that fail to meet the class 3 limits for the web are treated as class 4 sections (but the flanges should still meet the class 3 limits).

Lower width-to-thickness limits would apply to class 1 sections than apply to class 2 sections but since plastic global analysis is not used in bridge design, only the class 2 limits are of relevance.



Note: Strictly, the outstand width and the depth of the web should be measured clear of root fillets for rolled sections and welds for fabricated sections, but for simplicity the fillets or welds may be ignored.

Figure 5.2 *Parameters relevant to classification of cross section*

Flange outstand in compression

	Class 1	Class 2	Class 3
Grade S275	*(see text)*	$c/t \leq 9.2$	$c/t \leq 12.9$
Grade S355	*(see text)*	$c/t \leq 8.1$	$c/t \leq 11.3$

Webs - depth in compression

	Class 1	Class 2	Class 3
Grade S275	*(see text)*	$d_{wc}/t \leq 35$	$d_{wc}/t \leq 52$
Grade S355	*(see text)*	$d_{wc}/t \leq 32$	$d_{wc}/t \leq 45$

NOTES:

(1) These limits for webs are slightly more conservative than the detailed provisions in EN 1993-1_1.
(2) The class 3 limits for webs do not apply for cases where the web is wholly in compression (which can happen with axial compression) - more onerous limits of 39 (S275) or 34 (S355) should be used in such cases (instead of 52 and 45).

Openings

Openings in webs or compression flanges should not be made in a section that is designed as class 1 or class 2.

Any opening in webs or compression flanges of class 3 and 4 cross sections should be framed and the stiffened sections designed for local load effects, although such design is outside the scope of these simplified rules.

6 SECTION PROPERTIES

6.1 Effective section for global analysis

Gross elastic section properties should be used for global analysis. No allowance need be made for shear lag.

6.2 Effective section for bending stress analysis

For class 1 and 2 cross sections, the gross plastic section properties may be used.

For class 3 cross sections, the gross elastic section properties should be used.

For class 4 cross sections EN 1993-1-5 provides detailed rules for determining effective section properties, allowing for local buckling, but these are inappropriate for simplified design. Instead, conservative values may be determined by calculating the area of web at the d_{wc}/t_w limit for a class 3 section (i.e. $52t_w^2$ for S275 or $45t_w^2$ for S355) and spreading it over the actual web depth for the cross section (this clearly gives a lesser thickness that would not comply with width/thickness limits but it can be used to calculate the second moment of area).

There need be no deduction for fastener holes. (In EN 1993, unfilled holes and holes in tension flanges are deducted, but holes normally occur only at bolted splices and do not affect the design of the principal members.)

7 RESISTANCE OF BEAM CROSS SECTIONS

7.1 Bending resistance

The design bending resistance at ULS is given by:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \text{ for class 1 or class 2 cross sections} \tag{7.1}$$

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el} f_y}{\gamma_{M0}} \text{ for class 3 cross sections} \tag{7.2}$$

$$M_{c,Rd} = \frac{W_{eff} f_y}{\gamma_{M0}} \text{ for class 4 cross sections} \tag{7.3}$$

The elastic moduli W_{el} and W_{eff} correspond to the fibre with the maximum elastic stress.

Note that where the plastic resistance is used for class 1 or 2 cross sections, evaluation of the maximum bending moment at SLS will be required, to ensure that it does not exceed the elastic bending resistance ($= W_{el}f_y$, since the partial

RT 1156 version 02, February 2011

factor on material strength is unity at SLS). Where the elastic resistance is used, the SLS check will not need to be evaluated because the design loading at SLS is less (see Part B, Section 1.3).

7.2 Shear resistance

Generally, the design shear resistance of a web is given by:

$$V_{\mathrm{b,Rd}} = V_{\mathrm{bw,Rd}} + V_{\mathrm{bf,Rd}} \text{ but} \leq \frac{f_{\mathrm{yw}} h_{\mathrm{w}} t_{\mathrm{w}}}{\sqrt{3}\,\gamma_{\mathrm{M1}}} \tag{7.4}$$

where:

$V_{\mathrm{bw,Rd}}$ is the contribution from the web

$V_{\mathrm{bf,Rd}}$ is the contribution from the flange

h_{w} is the height of the web (the distance between the flanges)

t_{w} is the thickness of the web

f_{yw} is the yield strength of the web

7.2.1 Contribution from the web

The contribution from the web depends on its slenderness and thus on its tendency to buckle under shear loading. Usually in bridges transverse web stiffeners are provided at supports and at intermediate positions (intermediate transverse stiffeners are very effective in increasing the shear buckling resistance of a slender web by restricting the size of the rectangular panels within which buckling occurs - see further discussion below).

The non-dimensional slenderness of a web panel with intermediate transverse stiffeners is given by:

$$\overline{\lambda}_{\mathrm{w}} = \frac{h_{\mathrm{w}}}{34.4 t_{\mathrm{w}} \sqrt{k_{\tau}}} \text{ for S275 and } \overline{\lambda}_{\mathrm{w}} = \frac{h_{\mathrm{w}}}{30.3 t_{\mathrm{w}} \sqrt{k_{\tau}}} \text{ for S355 steel} \tag{7.5}$$

The parameter k_{τ} is a buckling coefficient that depends on the aspect ratio of the panel - i.e. the ratio of its depth hw to its width (between transverse stiffeners). The value of k_{τ} is given by the following expressions and by Figure 7.1:

$$k_{\tau} = 5.34 + 4.00 (h_{\mathrm{w}}/a)^2 \text{ when } a/h_{\mathrm{w}} \geq 1$$

$$k_{\tau} = 4.00 + 5.34 (h_{\mathrm{w}}/a)^2 \text{ when } a/h_{\mathrm{w}} < 1$$

Clearly, from the Figure, reducing the panel width (by reducing the spacing of intermediate transverse web stiffeners) can increase the buckling coefficient significantly. In multi-girder construction the aspect ratio is typically between 1.5 and 1.0 for regions of high shear. In ladder decks, the aspect ration may be 2.0 or more, although the web is thicker and thus its slenderness is lower.

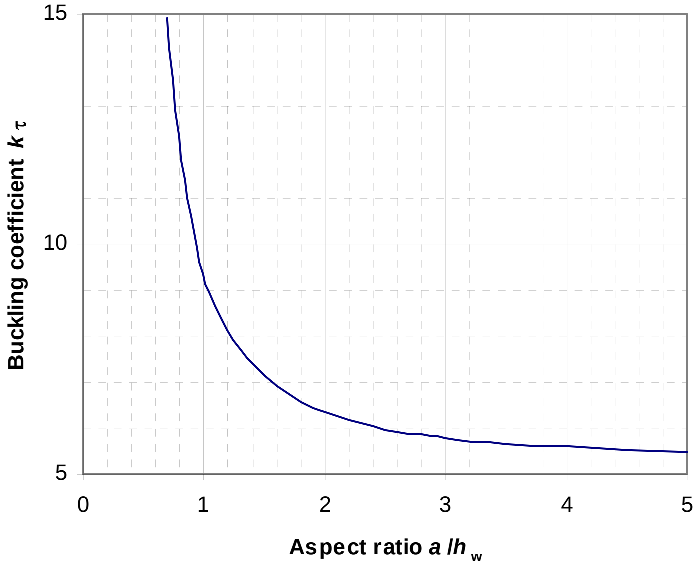


Figure 7.1 *Shear buckling coefficient k_τ*

From the slenderness, a reduction factor χ_w is derived, according to expression (7.6) and Figure 7.2.

$$\chi_w = 1.0 \text{ for } \bar{\lambda}_w \leq 0.83$$
$$\chi_w = 0.83/\bar{\lambda}_w \text{ for } 0.83 < \bar{\lambda}_w \leq 1.08 \qquad (7.6)$$
$$\chi_w = 1.37/(0.7 + \bar{\lambda}_w) \text{ for } 1.08 < \bar{\lambda}_w$$

These expressions apply where there is continuity of the web either side of the panel - at an end support there should be a 'rigid end post' (at the support the web continues beyond the centreline of the bearing and an endplate stiffener is provided - see Figure 7.3).

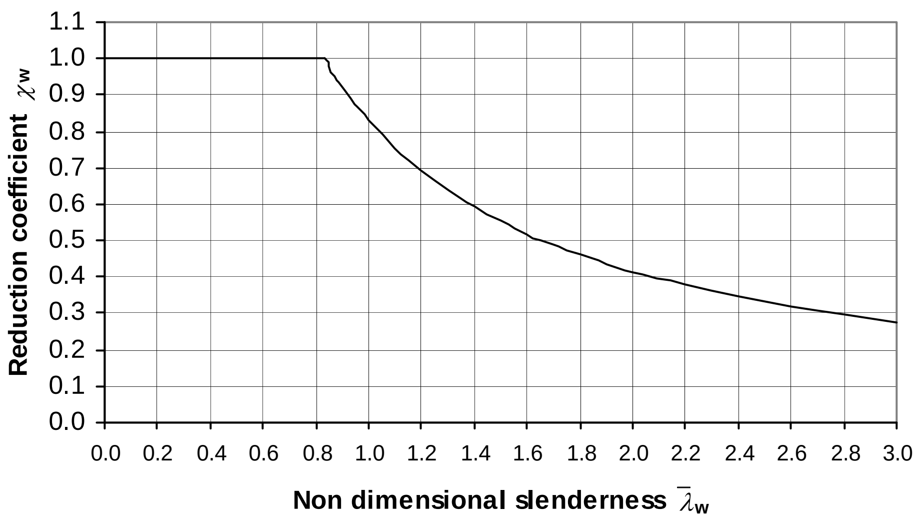


Figure 7.2 *Shear reduction factor χ_w*

The contribution of the web is then given by:

$$V_{\mathrm{bw,Rd}} = \chi_{\mathrm{w}} \frac{f_{\mathrm{yw}} h_{\mathrm{w}} t_{\mathrm{w}}}{\sqrt{3}\, \gamma_{\mathrm{M1}}} \qquad (7.7)$$

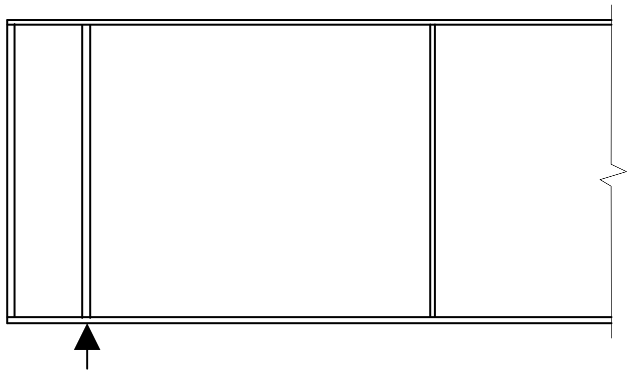


Figure 7.3 *End post detail at an end support*

7.2.2 Contribution from the flange

The contribution from the flange depends on the restraint from the local bending resistance of the flange to the tensile band that develops across the web panel. The value is given by the following expression in EN 1993-1-5:

$$V_{\mathrm{bf,Rd}} = \frac{b_{\mathrm{f}} t_{\mathrm{f}}^{2} f_{\mathrm{yf}}}{c\, \gamma_{\mathrm{M1}}} \left(1 - \left(\frac{M_{\mathrm{Ed}}}{M_{\mathrm{f,Rd}}} \right)^{2} \right) \qquad (7.8)$$

Where b_{f} and t_{f} are the width and thickness of the flange (use the values for the smaller flange), c is a factor related to the panel width a, M_{Ed} is the coexisting design moment acting on the section and $M_{\mathrm{f,Rd}}$ is the bending resistance of a cross section comprising the flanges only.

This is a complex expression relating the contribution to the level of coexisting moment and is shown in the interaction diagram Figure 7.5 discussed below. However the contribution is modest in most cases and can be ignored in simplified design. (Alternatively, a conservative value for $M_{\mathrm{Ed}} = 0$ can be obtained by setting $c = 0.25a$.)

7.3 Discussion of the development of shear resistance in slender webs

Figure 7.4 shows the effects of shear in a rectangular plate whose edges are subjected to a shear stress. A square element whose edges are orientated at 45° to the plate edge experiences principal tensile stresses on two opposing edges and compressive stresses on the other two, as shown. On a thin web these compressive stresses would induce local buckling resulting in elongated waves orientated diagonally as indicated in the second part of the Figure.

As the applied shear stress is increased beyond the critical shear stress (τ_{cr}) the plate buckles elastically and retains little stiffness in the direction in which the compressive stress acts. However, the plate still retains stiffness and an ability to carry greater load in the tensile direction. The inclined buckles become progressively narrower and tend to align with the panel diagonal; load is effectively carried in tension along the direction of the buckles and this is often referred to as "tension field". (Alternatively, the post-buckling behaviour is referred to as a "rotated stress field", because the principal tensile stress tends to align with the panel diagonal, rather than at 45°.)

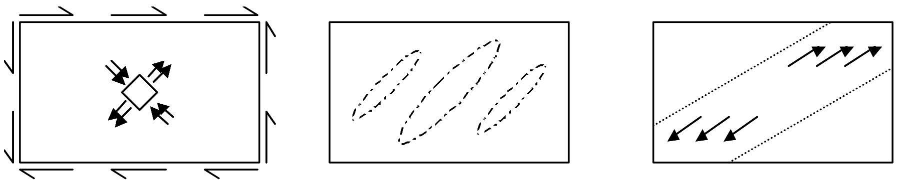


Figure 7.4 *Web Panels in shear: buckling and tension field action*

The shear resistance of the panel may be further enhanced by the local bending resistance of the flanges, which can be considered as providing boundary restraint to the tension band (this contribution is the $V_{\mathrm{bf,Rd}}$ parameter, as given by expression (7.8)).

7.4 Combined bending and shear

Under combined bending and shear, the values of the maximum moment within the panel, M_{Ed}, and the maximum shear force in the panel, V_{Ed}, should lie within the boundary shown on Figure 7.5. [NB the values to be considered are not necessarily at the same cross section, they are just the maximum values within the panel length.]

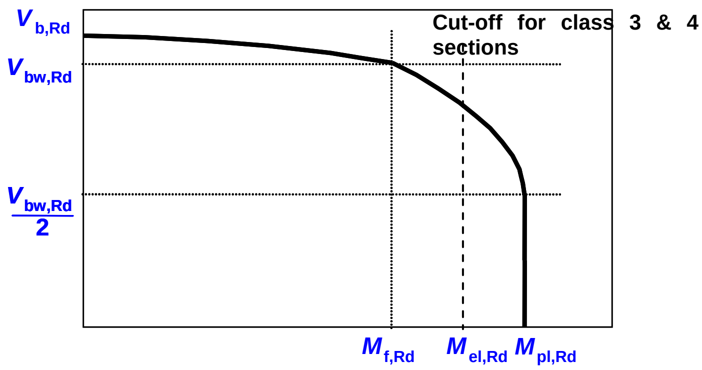


Figure 7.5 *Limiting interaction between moment and shear resistances*

In the diagram:

$V_{\mathrm{b,Rd}}$ is the shear resistance of the panel ($= V_{\mathrm{bw,Rd}} + V_{\mathrm{bf,Rd}}$)

$V_{\mathrm{bw,Rd}}$ is the contribution to the shear resistance from the web

$V_{\mathrm{bf,Rd}}$ is the contribution to the shear resistance from the flange

$M_{\mathrm{pl,Rd}}$ is the plastic bending resistance of the cross section (calculated regardless of the classification of the cross section)

$M_{\mathrm{el,Rd}}$ is the elastic bending resistance of the cross section

$M_{\mathrm{f,Rd}}$ is the bending resistance of the cross section ignoring the contribution from the web, and is calculated as:

$$M_{\mathrm{f,Rd}} = \frac{A_{\mathrm{f}} f_{\mathrm{y}} d_{\mathrm{f}}}{\gamma_{\mathrm{M0}}}$$

A_{f}, and f_{y} are the area and yield strength of the smaller flange

d_{f} is the distance between the centroids of the flanges.

In the Figure the shape of the limiting envelope between $M = 0$ and $M = M_{\mathrm{f,Rd}}$ is parabolic, given by expression (7.8). The shape between $M = M_{\mathrm{f,Rd}}$ and $M = M_{\mathrm{pl,Rd}}$ is also parabolic and the limiting value of M_{Ed} at a point on this part of the envelope is given by:

$$M_{\mathrm{lim}} = M_{\mathrm{pl,Rd}} - \left(M_{\mathrm{pl,Rd}} - M_{\mathrm{f,Rd}}\right)\left(\frac{2V_{\mathrm{Ed}}}{V_{\mathrm{bw,Rd}}} - 1\right)^2 \tag{7.9}$$

Note that the interaction envelope shown by the solid line is limited by the plastic moment resistance; where the section is class 3 or class 4 the same envelope applies but there is a cut-off at the elastic resistance, as shown by the dotted line.

For simplified design, the contribution of the flanges to shear resistance may be neglected and a linear interaction may be used, as shown in Figure 7.6. (The limit is given by a modified expression (7.9), without squaring the last term - this is easier to show graphically)

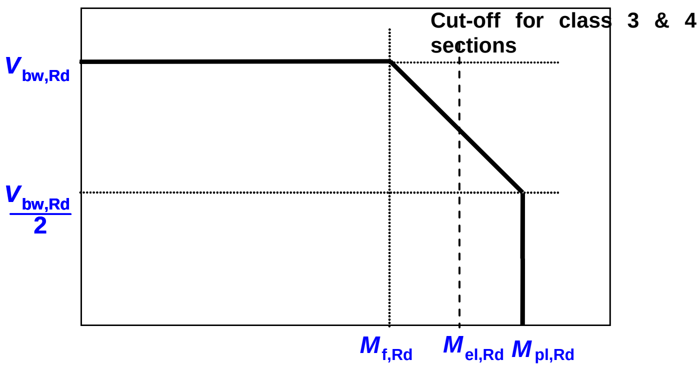


Figure 7.6 *Simplified interaction diagram*

Generally, for either interaction diagram, it may be noted that:

- Bending moments up to the value of $M_{\mathrm{pl,Rd}}$ or $M_{\mathrm{el,Rd}}$ (as appropriate) can be resisted by any beam if the shear force V_{Ed} does not exceed $0.5V_{\mathrm{bw,Rd}}$.
- Shear forces up to the value of $V_{\mathrm{bw,Rd}}$ can be resisted by any beam if the bending moment M_{Ed} does not exceed $M_{\mathrm{f,Rd}}$.

8 BUCKLING RESISTANCE OF BEAMS - LATERAL TORSIONAL BUCKLING

8.1 General

Lateral-torsional buckling is a phenomenon that can occur in a beam that is unrestrained transversely and can occur with the beam subjected to bending moments below the idealised capacity of the cross section. Figure 8.1 shows a simply supported beam having a deep I section and loaded vertically, thus producing bending about its major axis. It may also move laterally and twist, as shown.

The condition for buckling is generally expressed in terms of the elastic critical value of the maximum bending moment. This value depends on several geometric parameters – the beam length, support conditions and pattern of loading, the lateral stiffness, the torsional properties and the warping resistance of the section. The actual buckling resistance of the beam depends on this elastic value and on its strength (expressed as its cross sectional resistance).

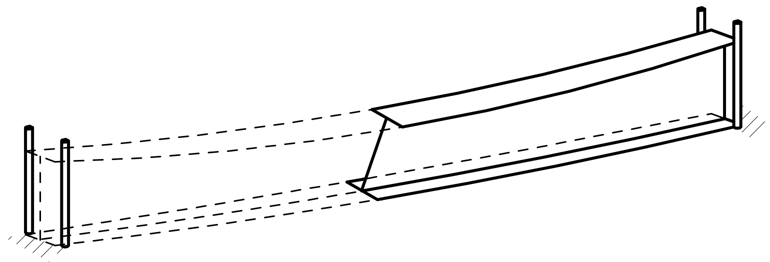


Figure 8.1 *Lateral torsional buckling of a simply supported beam*

In EN 1993, the buckling resistance is expressed in terms of a non-dimensional slenderness parameter $\bar{\lambda}_{\mathrm{LT}}$ that determines a reduction factor χ_{LT} which is then applied to the cross sectional bending resistance.

8.1.2 Non-dimensional slenderness

The non-dimensional slenderness parameter $\bar{\lambda}_{\mathrm{LT}}$ is given by:

$$\bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{W_{\mathrm{y}} f_{\mathrm{y}}}{M_{\mathrm{cr}}}} \tag{8.1}$$

where

W_y is the section modulus appropriate to the section class (see Section 7.1)

f_{y} is the yield strength of the compression flange

M_{cr} is the elastic critical moment for lateral torsional buckling

However, EN 1993 does not give expressions for M_{cr} and complementary information has to be used to derive it. The evaluation of M_{cr} for an ordinary mono-symmetric I section under uniform moment is quite complex, involving numerous parameters including the warping constant; evaluation for situations where the moment varies along the beam adds a further level of complexity. Instead of this complex evaluation, more simple (but conservative) procedures have been developed for evaluating $\bar{\lambda}_{\mathrm{LT}}$ for various configurations. Some of these procedures are given in Section 9 below.

8.1.3 Reduction factor

The reduction factor for lateral torsional buckling is given by expression (8.2) and by Figure 8.2.

$$\chi_{\mathrm{LT}} = \frac{1}{\phi_{\mathrm{LT}} + \sqrt{\phi_{\mathrm{LT}}^2 - \bar{\lambda}_{\mathrm{LT}}^2}} \text{ but } \chi_{\mathrm{LT}} \leq 1.0 \tag{8.2}$$

Where $\phi_{\mathrm{LT}} = 0.5\left[1 + \alpha_{\mathrm{LT}}\left(\overline{\lambda}_{\mathrm{LT}} - 0.2\right) + \overline{\lambda}_{\mathrm{LT}}^{2}\right]$ and α_{LT} is an imperfection parameter.

In EN 1993, four values of imperfection parameter are given and these define four 'buckling curves'. For simplified design, only one value of parameter need be considered, $\alpha_{\mathrm{LT}} = 0.76$. Figure 8.2 is drawn for that value.

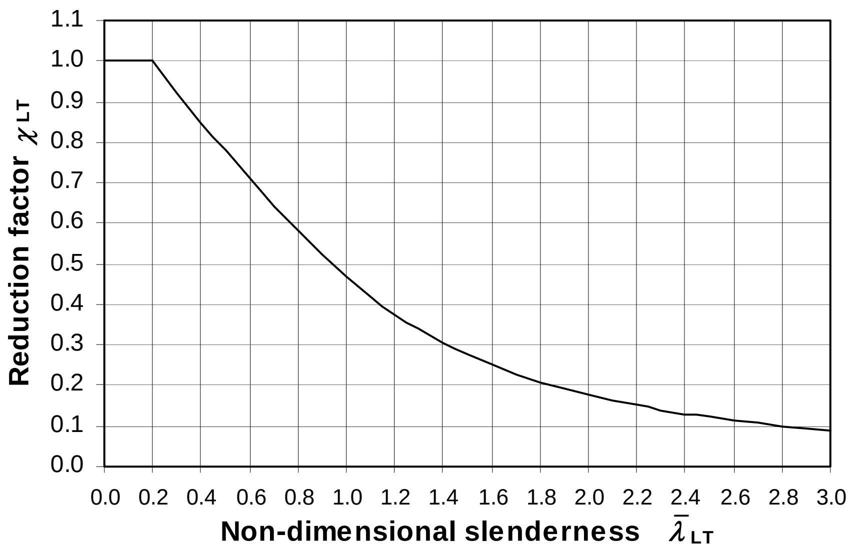


Figure 8.2 *Reduction factor χ_{LT}*

8.1.4 Buckling resistance

The design buckling resistance $M_{\mathrm{b,Rd}}$ is then determined simply by:

$$M_{\mathrm{b,Rd}} = \chi_{\mathrm{LT}} \frac{W_{\mathrm{y}} f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} \tag{8.3}$$

Where

W_y is the section modulus appropriate to the section class

Note that the γ_{M1} factor is applied, rather than the γ_{M0} factor that is applied in determining the bending resistance of the cross section, $M_{\mathrm{c,Rd}}$ (see Section 7.1).

9 SLENDERNESS OF BEAMS

As noted in Section 8.1.2, slenderness can be derived from some simplified expressions that avoid some of the complexity of the precise calculation of M_{cr}.

9.1 Beam segments between effective restraints

This section relates to lengths of steel beam between positions where there is torsional restraint and effective lateral restraint to the beam tat each end of the segment length. See further discussion of restraints in Section 10.

In such situations the slenderness may be taken from the following table:

Table 9.1 $\bar{\lambda}_{\mathrm{LT}}$ *for beams between effective restraints*

	S 275	S 355
Class 1 and 2	$\bar{\lambda}_{\mathrm{LT}} = \dfrac{L/i_{\mathrm{z,f}}}{87}$	$\bar{\lambda}_{\mathrm{LT}} = \dfrac{L/i_{\mathrm{z,f}}}{76}$
Class 3	$\bar{\lambda}_{\mathrm{LT}} = \dfrac{L/i_{\mathrm{z,f}}}{100}$	$\bar{\lambda}_{\mathrm{LT}} = \dfrac{L/i_{\mathrm{z,f}}}{88}$

L is the distance between points of restraint of the compression flange

$i_{\mathrm{z,f}}$ is the radius of gyration about the z-z axis of the gross cross-section of the compression flange plus one third of the height of the web in compression.

The above simplification takes no advantage of any variation of bending moment along the length of the span (i.e. it is the slenderness of a beam subject to uniform moment).

9.2 Beams without intermediate restraints

If there is no intermediate restraint to beams between support positions, it would be economic to take advantage of the variation of bending moment for the length between supports (which is not taken in the simplification in Section 9.1) but then a more detailed evaluation must be made. In such cases the slenderness for a straight uniform beam is given by the following expression:

$$\bar{\lambda}_{\mathrm{LT}} = \frac{1}{\sqrt{C_1}} V \frac{\lambda_{\mathrm{z}}}{\lambda_1} \sqrt{\beta_{\mathrm{w}}} \qquad (9.1)$$

where 1

C_1 is a parameter dependent on the shape of the bending moment diagram, such that the value of M_{cr} for the segment with the actual bending moment diagram is equal to C_1 times that for the same segment with a uniform bending moment along its full length. Values of $\dfrac{1}{\sqrt{C_1}}$ are given in Table 9.3.

(Conservatively, C_1 may be taken as 1.0.).

V is a parameter related to the slenderness and section geometry, and is given by Table 9.2, which is derived from the following expression:

$$V = \left[\left\{ 4i(1-i) + 0{,}05\lambda_{\mathrm{F}}^{2} + \psi_{\mathrm{i}}^{2} \right\}^{0,5} + \psi_{\mathrm{i}} \right]^{-0,5} \text{, in which}$$

$\psi_i = 2i - 1$ when $I_{\mathrm{z,c}} < I_{\mathrm{z,t}}$ and $\psi_i = 0.8(2i-1)$ when $I_{\mathrm{z,c}} \geq I_{\mathrm{z,t}}$.

For these expressions and for the Table, the following two parameters are required:

$$i = \frac{I_{\mathrm{z,c}}}{I_{\mathrm{z,c}} + I_{\mathrm{z,t}}}$$

$$\lambda_{\mathrm{F}} = \frac{L}{i_z} \cdot \frac{t_{\mathrm{f}}}{h}$$

h is the depth of the cross section;

L is the span of the beam

t_f is the mean thickness of the two flanges of the section

$I_{z,c}$, $I_{z,t}$ are the second moments of area of the compression and tension flanges, respectively, about their z-z axes.

$\lambda_z = \dfrac{L}{i_z}$

i_z is the radius of gyration of the section about the minor axis

$\beta_w = \dfrac{W_y}{W_{pl,y}}$

W_y is the modulus used to calculate $M_{b,Rd}$

For Class 1 and 2 sections $W_y = W_{pl,y}$

For Class 3 sections $W_y = W_{el,y}$

For Class 4 sections $W_y = W_{eff,y}$

$\lambda_1 = \pi\sqrt{\dfrac{E}{f_y}}$ in which f_y is the yield strength of the steel

($\lambda_1 = 87$ for S275 and $\lambda_1 = 76$ for S355 steel)

Table 9.2 *Parameter V for beams of uniform section*

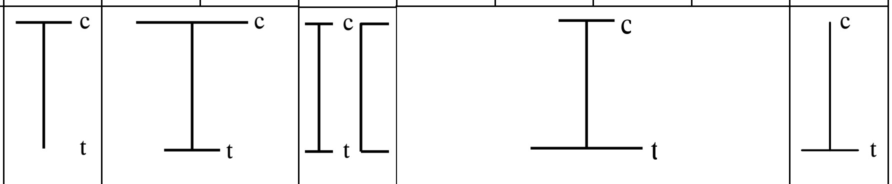
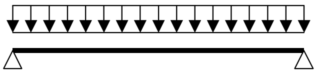
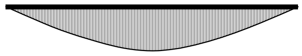
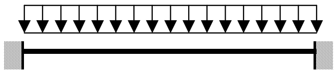
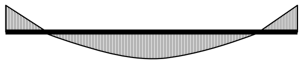
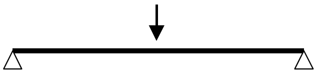
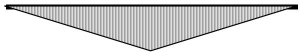
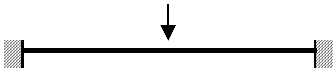
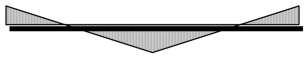
i	1.0	0.8	0.6	0.5	0.4	0.3	0.2	0.1	0
λ_F									
0.0	0.791	0.842	0.932	1.000	1.119	1.291	1.582	2.237	∞
1.0	0.784	0.834	0.922	0.988	1.102	1.266	1.535	2.110	6.364
2.0	0.764	0.813	0.895	0.956	1.057	1.200	1.421	1.840	3.237
3.0	0.737	0.784	0.859	0.912	0.998	1.116	1.287	1.573	2.214
4.0	0.708	0.752	0.818	0.864	0.936	1.031	1.162	1.359	1.711
5.0	0.679	0.719	0.778	0.817	0.878	0.954	1.055	1.196	1.415
6.0	0.651	0.688	0.740	0.774	0.824	0.887	0.966	1.071	1.219
7.0	0.626	0.660	0.705	0.734	0.777	0.829	0.892	0.973	1.080
8.0	0.602	0.633	0.674	0.699	0.736	0.779	0.831	0.895	0.977
9.0	0.581	0.609	0.645	0.668	0.699	0.736	0.780	0.832	0.896
10.0	0.562	0.587	0.620	0.639	0.667	0.699	0.736	0.779	0.831
12.0	0.528	0.549	0.576	0.591	0.613	0.638	0.665	0.697	0.733
14.0	0.499	0.517	0.539	0.552	0.570	0.589	0.611	0.635	0.662
16.0	0.474	0.490	0.509	0.519	0.534	0.550	0.567	0.586	0.607
18.0	0.452	0.466	0.482	0.492	0.504	0.517	0.531	0.547	0.564
20.0	0.433	0.446	0.460	0.468	0.478	0.489	0.502	0.515	0.529
NOTE 1 Configurations that give values of v greater than 2.0 should not be used in simplified design									

Table 9.3 *Values of* $\frac{1}{\sqrt{C_1}}$ *for cases with transverse loading*

Loading and support conditions	Bending moment diagram	$\frac{1}{\sqrt{C_1}}$
		0.94
		0.62
		0.86
		0.77

9.3 Beams with intermediate torsional restraints

Where there is no plan bracing within a span but there is intermediate torsional restraint (see Section 10.2 for a description of intermediate torsional restraint) the slenderness is reduced from what it would be for the unrestrained span. Determination of the effective slenderness of such bracing systems is complex and is best carried out using a computer model. It is also only relevant to the construction condition and therefore will not normally need to be evaluated in a student project.

However, if it is desired to check this construction condition for a student project, a modified version of expression (9.1) may be used:

$$\bar{\lambda}_{\mathrm{LT}} = V \frac{\lambda_{\mathrm{z}}}{\lambda_1} \sqrt{\beta_{\mathrm{w}}} \qquad (9.2)$$

where

V is as defined in Section 9.2 and calculated taking L as the span

β_w and λ_1 are as defined in Section 9.2

$\lambda_{\mathrm{z}} = \frac{L/3}{i_{\mathrm{z}}}$ (The use of $L/3$ rather than L is appropriate for multi-girder configurations where there at least 4 planes of cross bracing within the span and for ladder deck construction; in practice the value could well be between about $0.5L$ and $0.25L$.)

10 RESTRAINTS TO BEAMS

10.1 Effective intermediate lateral restraints

One means to improve the resistance to lateral torsional buckling of a slender beam is to provide effective intermediate restraints to the compression flange. For bare steel beams, effective restraint is provided by either plan bracing or by direct restraint from another structure (such as an adjacent deck slab). Restraint of the compression flanges of composite sections are discussed in Section 6 of Part D.

In the bare steel stage, an effective restraint system can be achieved by a triangulated plan bracing between a pair of main girders, just below the top flange; in such cases the slenderness can be taken as given by Table 9.1. To be 'fully effective' the lateral stiffness (displacement per unit force) must have a value that is at least $40EI_{z,c}/L^3$ where L is the length of the segments either side of the restraint and $I_{z,c}$ is the total second moment of area of the two compression flanges that are restrained by the bracing. Triangulated bracing is sufficiently stiff to provide this degree of restraint and its stiffness need not be checked in simplified design

To carry the forces associated with the restraining function, the bracing system or any individual restraint should be capable of resisting a force F_R plus the effects of any wind or other lateral forces. The force F_R is given by:

$$F_R = \frac{\Sigma N_{Ed}}{80} \tag{10.1}$$

where:

ΣN_{Ed} is the sum of the greatest forces in the compression flanges of the two beams connected by the bracing.

10.2 Torsional restraints

The provision of plan bracing to the top flange complicates the construction of the deck slab. (Plan bracing above the flange effectively prohibits the use of precast permanent formwork, because the units cannot be placed once the bracing is in position; bracing below the flange may need to be removed after concreting the slab, to avoid maintenance and potential fatigue problem). Instead, reliance is often placed in the construction condition on 'torsional restraints'. Such restraint takes the form of triangulated planes of bracing between a pair of beams, with no plan bracing. (The bracing later becomes effective restraint to the bottom flanges adjacent to supports, after the slab has been cast.) The LTB mode of buckling then involves the vertical displacement of the beams and it is their stiffness in bending that provides restraint to LTB, although such restraint is not 'fully effective'. The mode of buckling is illustrated in Figure 10.1. In ladder deck construction, the presence of the cross beams also achieves torsional restraint, although the stiffness is reduced slightly by the flexibility of the cross beams in bending.

RT1156 version 02, February 2011

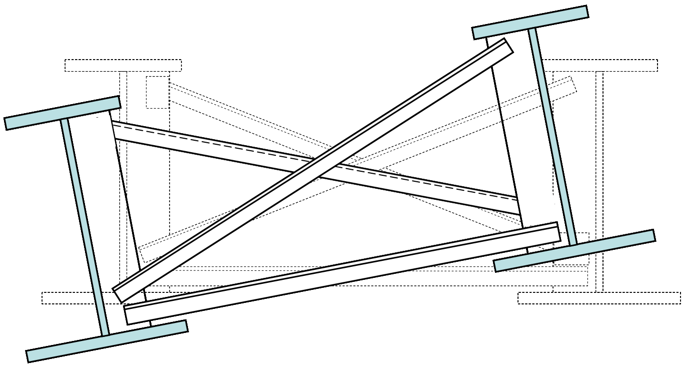


Figure 10.1 *Buckling mode for paired beams - torsional restraints*

If the effects of such torsional restraints are to be considered in a student design, a simplified method of determining a value of non-dimensional slenderness is given in Section 9.3 above.

10.3 Restraints at supports

All beams should be restrained against rotation about their longitudinal axes at each support. The bracing system should also be capable of transferring any lateral forces (e.g. forces due to wind) to bearings which provide lateral restraint to the bridge.

The restraint system at a support should be able to resist a couple comprising opposing forces F_{R}, as given by expression (10.1), in conjunction with any lateral load.

A triangulated bracing system or a full-depth cross beam at a support may be assumed to provide a sufficiently stiff and strong restraint for simplified design.

11 INTERMEDIATE TRANSVERSE WEB STIFFENERS

11.1 General

Intermediate transverse web stiffeners are provided to enhance the shear capacity of webs. The usual form of stiffener is a simple flat (complying with the shape limitations of Section 11.2) welded to the face of the web. This effectively creates a T-shaped beam over the depth of the web.

11.2 Geometric proportions

Flat plates welded to the face of the web should be used to create intermediate transverse web stiffeners. The proportions of the plate outstand should be limited as follows:

$$\frac{h_{o}}{t_{s}} \leq 12.0 \text{ for S275 and } \frac{h_{o}}{t_{s}} \leq 10.5 \text{ for S355} \qquad (11.1)$$

Where h_{o} is the height of the outstand and t_{s} is the thickness of the stiffener.

Stiffeners outstands are typically just less than the outstand of the narrower flange, though the required size depends on criteria for stiffness and strength. 'Rounded dimensions' (e.g. 150×15, 200×20, 250×25) are normally specified.

11.3 Effective section of transverse web stiffener

The effective stiffener section should comprise the stiffener, along with a portion of web plate of width be as shown in Figure 11.1.

The value of b_e is $13.8t_w$ for S275 steel or $12.2t_w$ for S355 steel, but not more than $a/2$, where a is the width of the panel on that side.

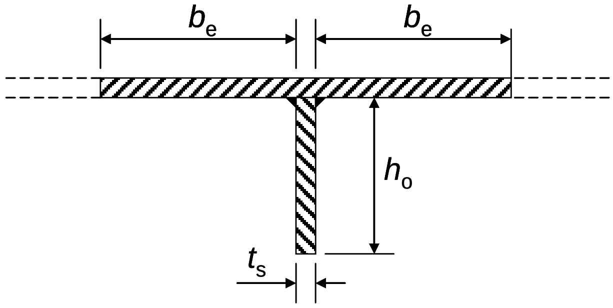


Figure 11.1 *Effective stiffener section*

11.4 Loading on transverse web stiffeners

Intermediate transverse web stiffeners should be designed to resist a force P_{Ed}, given by:

$$P_{Ed} = V_{Ed} - h_w t_w \times 0.8\tau_{cr}\sqrt{1-\frac{\sigma_{x,Ed}}{0.8\sigma_{cr,x}}} \quad \text{when a} \geq h_w$$

$$P_{Ed} = \left[V_{Ed} - h_w t_w \times 0.8\tau_{cr}\sqrt{1-\frac{\sigma_{x,Ed}}{0.8\sigma_{cr,x}}}\right]\frac{a}{h_w} \quad \text{when a} < h_w$$

where:

τ_{cr} $= k_\tau \sigma_E$

$\sigma_{cr,x}$ $= 4\ \sigma_E$

k_τ is the buckling coefficient given in 7.2.1

σ_E $= 190000\left(\frac{t_w}{h_w}\right)^2$

$\sigma_{x,Ed}$ is the compressive direct stress in the web (for a symmetric section $\sigma_{x,Ed} = 0$ but for asymmetric sections, $\sigma_{x,Ed}$ is the algebraic mean of the stresses at the top and bottom of the web, taken as zero if tensile).

The force arises in the plane of the web and causes both axial and bending stresses in the stiffener.

11.5 Strength and stiffness of transverse web stiffeners

The strength of the stiffener is verified using the simplified interaction criterion:

$$\frac{N_{Ed}}{N_{b,Rd}} + \frac{M_{Ed}}{M_{b,Rd}} \leq 0.9$$

Where $N_{Ed} = P_{Ed}$ (from 11.4) and M_{Ed} is due to the eccentricity of P_{Ed}

$$N_{b,Rd} = \frac{\chi A_s f_y}{\gamma_{M1}}$$

A_s is the cross sectional area of the effective stiffener section

χ is a reduction factor for flexural buckling (see Section 15 for the derivation of this factor from a non-dimension slenderness, taking $L_{cr} = h_w$.

$$M_{b,Rd} = \frac{W_{el} f_y}{\gamma_{M1}}$$ (there is no reduction for slenderness, because of the restraint from the web -plane)

12 BEARING STIFFENERS AT SUPPORTS

12.1 General

Webs of plate girders and rolled beams should be provided with a system of load bearing stiffeners at each support position.

The stiffeners should be symmetrical about the web. Flat plates welded to the faces of the web are usually used, which creates a cruciform shaped effective section. Two stiffeners are often provided on each face, either to carry the reaction without the need for very thick plates or to cater for longitudinal eccentricity of the reaction from a sliding bearing. Typical arrangements are shown in Figure 12.1.

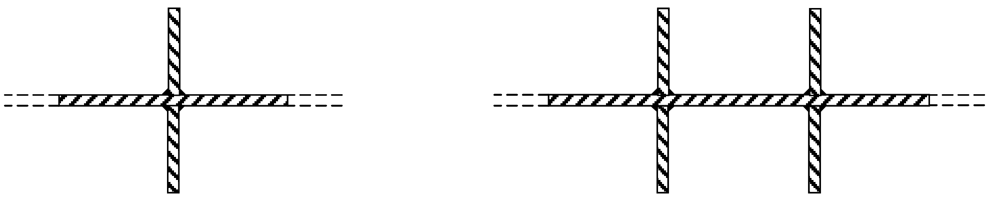


Figure 12.1 *Bearing stiffener arrangements*

12.2 Effective section for bearing stiffeners

The same limitations on outstand to thickness ratio apply as for intermediate stiffeners and the same proportion of web may be considered as part of the effective section.

For double leg stiffeners, the spacing between the two legs should not exceed double the value of b_e given in Section 11.3 (or the legs would have to be treated as independent stiffeners).

12.3 Forces on bearing stiffeners

The force in a bearing stiffener is equal, at the bottom, to the vertical reaction on the bearing, plus (potentially) biaxial bending due to the eccentricity of the reaction from the centroid of the effective stiffener section. In practice, designers would allow a nominal eccentricity in each direction, plus an eccentricity due to thermal expansion/contraction at a sliding bearing. These eccentricities do not normally need to be considered in student projects.

12.4 Strength of bearing stiffeners

The resistance of a bearing stiffener to combined axial load and biaxial bending needs to be verified as does its buckling resistance.

For resistance of the cross section, a simple linear interaction between the utilization of axial and bending resistances may be assumed, which may be expressed as:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{c,Rd}}}+\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{c,y,Rd}}}+\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{c,z,Rd}}}\leq 1 \qquad (12.1)$$

Where

N_{Ed} $M_{\mathrm{y,Ed}}$ and $M_{\mathrm{z,Ed}}$ are the design axial force and moment about each axis

$N_{\mathrm{c,Rd}}$ $M_{\mathrm{c,y,Rd}}$ and $M_{\mathrm{c,z,Rd}}$ are the design resistances of the stiffener cross section

(The y-y axis lies in the web and the z-z axis is perpendicular to the web.)

(Or, perhaps more simply, calculate the total stress at ULS due to axial force and elastic bending at an extreme fibre and ensure that it does not exceed the design yield strength).

For buckling resistance EN 1993-1-1 provides a complex interaction relationship for members subject to combined compression and bending but these are difficult to apply to the usual configuration of a bearing stiffener.

For simplified design interaction of buckling effects may be ignored, provided that the flat stiffeners have an outstand of at least 10% of the depth of the girder (in which case the ‘column’ between the two flanges is stocky and its resistance is not reduced by flexural buckling).

13 WELDED CONNECTIONS

13.1 General

Welding offers a means of making continuous, load-bearing metallic joints between the components of a structure. All welded joints can be typified by or made up from four configurations. These are:

1. In-line butt
2. Tee
3. Lap
4. Corner

Fillet welds are generally used for configurations 2 and 3.

13.2 Butt welds

For simplified design, butt welds should be full strength butt welds. A full strength butt weld achieves full penetration through the thickness of the welded material.

13.3 Fillet welds

A fillet weld is applied outside the surface profile of plates. Thus, the joint may be formed either by the overlapping of members of by the use of secondary joint material.

End returns

A fillet weld should be returned continuously around the corner of the end or the side of a part, for a length beyond the corner of not less than twice the leg length of the weld.

Effective throat of a fillet weld

The effective throat of a fillet weld a is the height of a triangle that can be inscribed within the weld and measured perpendicular to its outer side (see Figure 13.1). For fillet welds between faces at right angles the effective throat a is equal to the leg length divided by $\sqrt{2}$.

It has been traditional UK practice to specify (on drawings etc.) the leg length of a fillet weld but continental European practice is to specify throat thickness. When specifying fillet weld size, make sure that it is clear which dimension is specified note that if throat dimension is given but mistakenly interpreted as leg length, the weld would be substantially undersized.

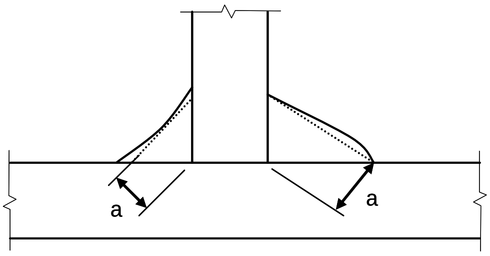


Figure 13.1 *Effective throat of a fillet weld*

Minimum size of fillet weld

The minimum throat thickness for a fillet weld is 3 mm (or minimum leg length of 5 mm).

Effective length of a fillet weld

The effective length of a fillet weld may be taken as its overall length, provided that the end returns mentioned above have been specified.

Design resistance of a fillet weld

The design shear strength of a fillet weld is given by:

$$f_{\mathrm{vw,d}} = \frac{f_{\mathrm{u}} / \sqrt{3}}{\beta \gamma_{\mathrm{M2}}} \qquad (13.1)$$

Where

f_{u} is the nominal ultimate tensile strength of the weaker part joined (= 430 N/mm^2 for non-alloy S275 steel, = 510 N/mm^2 for non-alloy and weathering S355 steel)

β_{w} is a correlation factor (= 0.85 for S275 steel, = 0.90 for S355)

Thus $f_{\mathrm{vw,d}}$ = 234 N/mm^2 for S275 and = 262 N/mm^2 for S355.

The design resistance of a fillet weld per unit length is then given by:

$$F_{\mathrm{w,d}} = f_{\mathrm{vw,d}}\, a \qquad (13.2)$$

RT 1156 version 02, February 2011

This value, based on the shear strength, may be used (conservatively) independently of the orientation of the force on the weld to the weld axis. (For welds that are subject to combined shear and forces normal to the weld axis EN 1993-1-8 provides a more general evaluation but the above is adequate for simplified design in all cases.)

14 BOLTED CONNECTIONS

14.1 General

Bolted connections in bridges are made using preloaded bolts (commonly referred to as high strength friction grip bolts). These bolts are tightened in such a way that a reliable preload is achieved in the bolt. This preload allows shear to be transferred between the interfaces (faying surfaces) by friction – i.e. the surfaces do not slip, one relative to another, until the frictional resistance has been overcome.

Such bolts are used to ensure that there is no movement (slip) at a bolted joint under normal (i.e. serviceability limit state) conditions. In most connections it is acceptable to allow the bolts to slip at the higher ULS loads, at which time they act in bearing and shear (this is usually significantly greater than the ULS friction capacity).

For simplified design in undergraduate projects, the following rules are given, together with typical design values. It should be noted that the requirement for no slip at SLS will normally be more onerous than for bearing/shear resistance at ULS.

14.2 Girder splices

Girders are usually connected using 'double cover plates'. See Figure 14.1. These are additional plates, one on each side of the element to be spliced. The bolts are then in 'double shear'. Bracing is usually connected simply by lapping, in which case the bolts are in single shear.

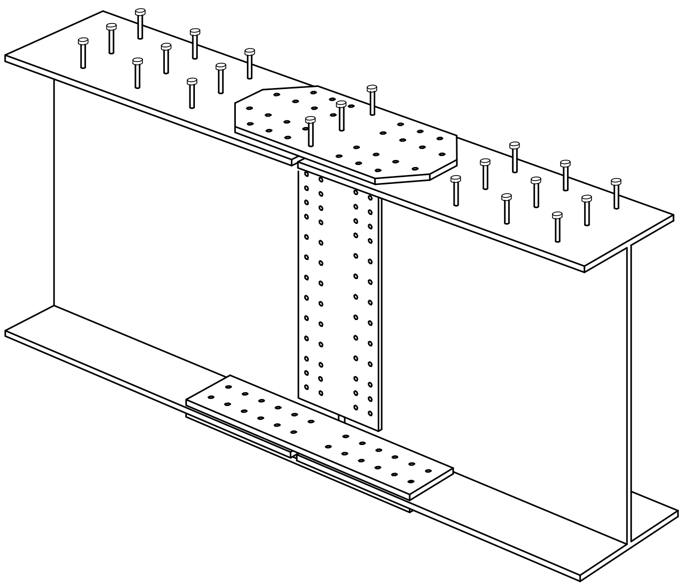


Figure 14.1 *Girder splice, showing cover plates and shear studs*

14.3 Cross girder connections in ladder decks

The intermediate cross girders in ladder deck bridges are usually connected to the main girders using a simple lapped connection of the web of the cross girder onto a transverse web stiffener. This detail is illustrated in Figure 14.2. Cross girders at supports are usually connected using double web cover plates (as in a girder splice) but that connection detail would not usually need to be considered in a simplified design.

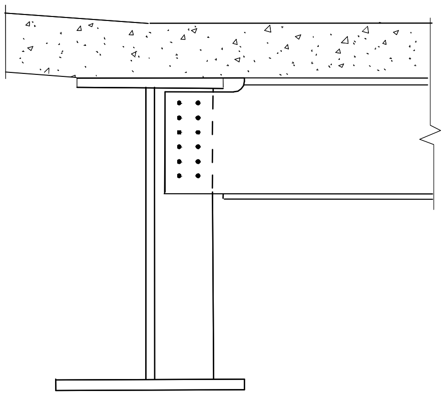


Figure 14.2 *Connection of intermediate cross girder in a ladder deck bridge*

The cross girder connections are usually designed against slip at ULS. For a student project, if such a connection is to be designed in outline, the number of bolts required may be determined from the shear at the end of the cross girder and the provision of sufficient slip resistance at ULS in single shear to carry 150% of the vertical shear force (this makes an allowance for combined shear and moment transferred through the connection).

The reason for designing for no slip at ULS for these connections is that the U-frame action of cross girder and web stiffeners is relied upon to restrain the bottom flange in hogging moment regions; slip before ULS would compromise this restraint.

14.4 Slip resistant connections

The design slip resistance of a preloaded bolted connection (HSFG bolted joint) is given by:

$$F_{s,Rd} = \frac{k_s n \mu F_{p,c}}{\gamma_{M3}} \tag{14.1}$$

where

$F_{p,c}$ is the preload in the bolt

n is the number of friction surfaces

μ is the slip factor

k_s is a factor related to the type of hole (normal, oversize, etc.)

Typical friction values at SLS, for M24 and M27 bolts grade 8.8 in normal clearance holes, and for grit blasted steel surfaces, are given in Table 14.1.

Table 14.1 *Slip resistance of preloaded connections*

	SLS capacity (KN)		ULS capacity (KN)	
	Double shear	Single shear	Double shear	Single shear
M24	180	90	158	79
M27	234	117	206	103

14.5 Shear resistance of bolted connection

Where a bolt does slip into bearing and shear, its shear resistance is given by:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} \text{ per shear plane} \tag{14.2}$$

Where

f_{ub} is the ultimate tensile strength of the bolt

A is the tensile area of the bolt

α_v = 0.6 for grade 8.8 bolts and = 0.5 for grade 10.9 bolts

Typical shear capacities, for M24 and M27 bolts, grade 8.8, in normal clearance holes, are given in Table 14.2.

Table 14.2 *Shear values of HSFG bolted connections*

	ULS capacity (kN)	
	Double shear	Single shear
M24	271	136
M27	353	176

The above values apply where the shear planes pass through the unthreaded portion of the bolt, which is the usual design condition.

It should be noted that, for the thicknesses of plates that are normally used in bridges, the shear capacity is less than the bearing capacity. Consequently, limitations for bearing capacity are not given in this simplified document.

14.6 Cover plates

For simplified design, cover plates should be provided on both faces of the part being connected. No specific rules for plate thickness are given in EN 1993-1-8 but in normal practice the cover plates should be at least half as thick as the element joined and not less than 8 mm thick.

14.7 Spacing of bolts

There are limitations on the maximum and minimum spacing of bolts. The maximum spacing depends on the thickness of the outer cover plates. The minimum spacing depends on the diameter of the bolt. For bolts arranged in an orthogonal pattern (lines of bolts parallel to the edges of the connected parts) a simplified table of maximum spacing is given in Table 14.3

Table 14.3 *Maximum spacing of bolts*

Cover plate (mm)	Maximum spacing (mm)
8	112
10	140
12	168
15 and over	200*

* For weathering steel the limit is 175 mm

The minimum spacing is 57 mm for M24 bolts and 66 mm for M27 bolts (use rounded minimum dimensions of 60 mm and 70 mm respectively).

15 CABLE STAYED BRIDGES

The design of cable stayed bridges generally requires more complex global analysis, possibly even including second order (large deflection) analysis that is beyond the scope of student project designs. However, cable stayed configurations are quite often proposed in project designs and some simplified guidance is appropriate.

Generally, for student project designs, second order analysis should be avoided. For most configurations that are likely to be considered the effects of the displacement on the load/displacement relationship is modest and may be ignored.

The only effect on the design of the deck structure is the addition of axial compression, due to the connection of the stays. This simplified design guide does not deal with combined bending and axial load on beams but a simple allowance can be made by ensuring that the combined utilization of axial and bending resistance does not exceed unity (e.g. if the axial compression is 10% of the axial resistance of the deck cross section, ensure that the bending does not exceed 90% of the bending resistance).

The cable stays themselves are in effect prestressed members. It is the intention that the required bridge geometry is achieved under full dead load (SLS characteristic values); at that stage the stays are carrying tensile loads and therefore have tensile strains. (The unstrained lengths would be shorter than the distances between anchorages in the permanent load condition.) The effect of this prestress can be achieved in computer models by imposing a negative thermal strain, of a value the same as the tensile strain under (SLS) dead load. This imposed strain is the P action referred to in expression (3.1) and thus a factor γ_P has to be applied to the imposed strain for the calculation of ULS forces and moments. The value of γ_P is not explicitly given in EN 1993-2 but the same factor should be applied as is applied to permanent actions at ULS.

Where stays are used, appropriate sizes and mechanical properties should be used for the stay members. The modulus of elasticity for bars and parallel wire strands may be taken as that for structural steel but for spiral strand and locked coil ropes the modulus should be taken as about 70% of that for structural steel (because of the way the spiral strand responds to axial tension).

The design value of the tension resistance of a stay member is given by:

$$F_{\mathrm{Rd}} = \frac{F_{\mathrm{uk}}}{1.5}$$

Where F_{uk} is the characteristic value of the breaking strength, which depends on the type of tension component:

For solid bars and stays comprising bundles of parallel wires:

$$F_{\mathrm{uk}} = A_{\mathrm{m}}\, f_{\mathrm{uk}}$$

Where

A_{m} is the metallic cross-sectional area ;

f_{uk} is the characteristic value of the tensile strength

For spiral strands terminated in filled sockets:

$$F_{\mathrm{uk}} = Kd^2 R_{\mathrm{r}}$$

Where

K is the minimum breaking force factor (= 0.50 for spiral strand rope and = 0.60 for fully locked coil rope)

d is the nominal diameter of the rope

R_{r} is the tensile strength of the wires in the strand (usually expressed in relation to "rope grade"

For full details of steel wire ropes, refer to BS EN 12385.

Compression members

If a cable stayed design is being carried out, the towers or pylons will also need to be designed. This simplified version of the Eurocodes is not intended for detailed design of compression members but the following brief guidance may be used conservatively.

The buckling resistance of a compression member is given by:

$$N_{\mathrm{b,Rd}} = \frac{\chi A f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}}$$

Where

χ is the reduction factor for buckling

A is the cross sectional area of the member

f_{y} is the characteristic yield strength

The gross area of the member may be used in most cases, assuming that the cross section is not Class 4. The classification of flange outstands is the same as for bending (see Section 5.2) but for 'internal elements' (a web or a flange of a box section) the limiting values for Class 3 sections are as given by Note 2 to the table for webs on page33.

The reduction factor for buckling depends on the slenderness of the member and on a buckling curve. The slenderness is given by:

$$\bar{\lambda} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$$

Where

L_{cr} is the buckling length in the plane considered

i is the radius of gyration about the relevant axis

λ_1 $= \pi\sqrt{\dfrac{E}{f_y}}$ = 76 for S355 grade steel

For simplicity, the same buckling curve as for lateral torsional buckling (see Figure 8.2) may be used to determine χ according to the value of $\bar{\lambda}$.

In most cases it will not be necessary to consider combined compression and bending but if it is, a linear interaction may be used:

$$\frac{N_{Ed}}{N_{b,Rd}} + k\frac{M_{Ed}}{M_{b,Rd}} \leq 1$$

For simplicity, k may be taken as 1.0.

Part D - Design of composite bridges (Simplified version of EN 1994-2)

(For use in student projects)

CONTENTS

RT 1156 version 02, February 2011

1 GENERAL

1.1 Scope of this Part

This Part presents rules for verifying the adequacy of composite beams in bridges. The rules in this Part only cover the verification of beam elements and the shear connection between the steel beams and the reinforced concrete slab.

The simplified procedures are based on rules in EN 1994-2. That Part of the Eurocode provides rules relating to composite action but refers to EN 1993-2 and to EN 1992-2 for particular aspects relating to steel and concrete elements respectively. This part of the simplified document thus refers to Part C for aspects of the design of steel elements; detailed design of reinforced concrete is only covered insofar as it is part of a composite section (there is no simplified version of EN 1992-2 in this document).

Fatigue design

It is usual to verify the fatigue endurance of the shear connection between the steel beams and the slab, and also the endurance of reinforcement that is subject to cyclic tensile stresses. However, as noted in Part C of this document, fatigue design is outside the scope of this simplified document.

1.2 Symbols

In addition to the symbols for variables noted in Section 1.2 of Part C, the following are used in the design of composite bridge structures:

f strength (of steel or concrete material)
n modular ratio
v shear per unit length or area
P force on or resistance of shear connector

Additional subscripts are

a related to steel
c related to concrete
S related to reinforcement

Thus:

f_{ck} is the characteristic concrete cylinder strength

f_{sy} is the characteristic yield strength of reinforcement

P_{Rd} is the design value of the shear resistance of a single connector

2 BASIS OF DESIGN

2.1 Ultimate limit state (ULS)

All beams should be designed to provide adequate strength to resist the design effects of the actions, using partial factors that are appropriate to the ULS. In addition to the considerations listed in Section 2.1 of Part C, the following need to be considered.

- The effects of shear lag in the concrete slab
- Time-dependent effects in concrete
- The non-linear stress-strain relationship for concrete

- The resistance to longitudinal shear provided by shear connectors
- Distortional buckling of cross sections

2.2 Serviceability limit state (SLS)

In addition to the requirements in Section 2.2 of Part C, the following need to be considered:

- The deflections should not cause the structure to infringe any clearance gauge
- The compressive stress in the concrete should be limited, for durability reasons
- The crack widths in reinforced concrete in tension should be limited, for durability

2.3 Partial factors

The values of partial factors to be used are given in Table 2.1 (see Part C for factors relating to structural steel).

Table 2.1 *Partial factors*

Partial factor on:		ULS	SLS
Resistance of concrete	γ_C	1.5	1.0
Resistance of reinforcing steel	γ_S	1.15	1.0
Resistance of headed shear stud	γ_V	1.25	1.0

These factors are given in the NA to EN 1992-1

3 MATERIAL PROPERTIES

3.1 Material properties for concrete

The characteristic strength of concrete in compression is based on the specified cylinder strength at 28 days. A typical concrete for bridges is grade C40/50, for which the cylinder strength is 40 N/mm^2 and the cube strength is 50 N/mm^2. For simplified design, this concrete grade should be used and then:

f_{ck} = 40 N/mm^2

E_{cm} = 35 000 N/mm^2

Where E_{cm} is the short-term modulus of elasticity.

EN 1992-1 gives an expression for determining the long-term modulus of elasticity of concrete but for simplified design only the short-term modulus needs to be considered - all design effects may be applied as short-term loading and thus the value of the long-term modulus is not needed.

3.2 Properties of reinforcing steel

For simplified design, it should be assumed that the reinforcing steel is ribbed bar, grade 500, to EN 10080. Thus the characteristic stress is:

f_{sk} = 500 N/mm^2

The modulus of elasticity is taken to be the same as that for structural steel.

3.3 Properties of structural steel

See Section 3 of Part C of this document.

4 ANALYSIS

4.1 Analysis of staged construction

It is usual for composite bridges to be constructed in stages, such that the steel beams initially carry the weight of wet concrete and the composite beams subsequently carry super-imposed dead loads and live loads. This form of construction may be evaluated by analysing separately loads on a model with bare steel beams and loads on a model with composite beams. The loading on the composite structure also distinguishes between long-term loading, for which the behaviour depends on the long-term modulus of elasticity, and short-term loading (essentially the variable loading, mainly traffic), for which the behaviour depends on the short-term modulus. The total effects at any cross section are thus the sum of three separate results.

For simplified design, no distinction need be made between long-term and short-term loading on composite beams. For even greater simplicity, at the composite stage all the loads may be assumed to be applied to the composite beams. (This is slightly non-conservative.) However, staged construction should always be considered and a check should be made on the bare steel beams acting alone during construction, when they are carrying the weight of the wet concrete, because that is frequently a prime consideration for the bridge constructors.

4.2 Section properties for global analysis

In carrying out the global analysis for the composite structure, the cross sectional properties should be determined using the transformed slab area (i.e. divided by the modular ratio) or, for cracked slabs, using the area of tensile reinforcement. EN 1994-2 requires that the effects of shear lag in the slab are taken into account in the section properties (this is done by using an effective breadth of slab, rather than its full width) but for simplified design, shear lag may be ignored.

Uncracked concrete should be assumed for all midspan regions and for simply supported spans. Cracked concrete should be assumed for a length of approximately 15% of the span either side of each intermediate support in continuous spans.

4.3 Global analysis

As for steel beams, elastic global analysis should be used to determine the load effects in the structure. In addition to permanent and variable loads, EN 1994-2 requires the evaluation of the effects of shrinkage in concrete and differential temperature (vertical temperature gradient). These effects are normally relative small in relation to the total effects but require a detailed analysis. Such analysis is outside the scope of this simplified design and the effects may be ignored in student project designs.

5 RESISTANCE OF BEAM CROSS SECTIONS

5.1 Design strengths of materials

To determine the design resistance of a composite section, design values of the material strengths (i.e. characteristic values divided by the appropriate partial factors) need to be determined, for use in the expressions that determine design resistance. Thus the expressions for design resistance differ from those for steel sections, which generally present the design resistance value in an expression where a geometric section property (area, modulus, etc,) is multiplied by a characteristic strength and then divided by a partial factor for material strength.

The (ULS) design strengths for concrete, reinforcing steel and structural steel are given by:

$f_{cd} = f_{ck} / \gamma_C$ for concrete in compression (5.1)

$f_{sd} = f_{sk} / \gamma_S$ for reinforcement in tension (5.2)

$f_{yd} = f_{yk} / \gamma_M$ for structural steel (use γ_{M0} or γ_{M1} as appropriate) (5.3)

5.2 Bending resistance

Class 1 and 2 cross sections

For a composite beam of class 1 or 2 cross section, the bending resistance may be based on a plastic distribution of stress, provided that there is full interaction between the steel and the concrete (this is achieved by designing the shear connection in accordance with Section 7 below).

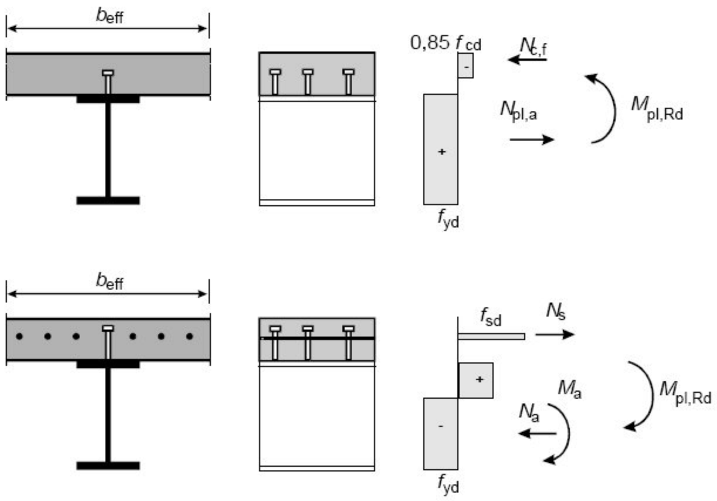


Figure 5.1 *Plastic stress distributions in a composite beam*

In the plastic stress blocks shown above, the following assumptions are made:

- The structural steel is stressed to its design yield strength in tension or compression
- Reinforcement in compression is ignored; reinforcement in tension is stressed to its design yield strength

- Concrete in tension is ignored; concrete in compression is stressed to 85% of its compressive design strength over its whole depth from the plastic neutral axis.

The position of the plastic neutral axis is such that the tensile and compressive forces balance (unless there is an axial force, which is a situation that is outside the scope of this simplified document).

The plastic resistance moment is calculated by summing the moments of these stress blocks about the plastic neutral axis.

Where the plastic moment resistance is utilised at ULS, the elastic moment resistance at SLS must also be checked.

Class 3 and 4 sections

For composite beams that are of class 3 or class 4 cross section, the bending resistance of the cross section should be calculated by elastic theory. The resistance to bending is the lowest value of moment that causes the concrete in compression, the reinforcement in tension or the steelwork in either tension or compression, to reach the design value of the material strength.

5.3 Resistance to vertical shear

Vertical shear should be assumed to be carried entirely on the steel section and should therefore be checked in accordance with Part C of this document.

6 BUCKLING RESISTANCE OF COMPOSITE BEAMS

For the bridges within the scope of this document, the top flanges of the girders are connected to the deck slab by shear connectors. The top flange is therefore fully braced against buckling its compressive regions.

In continuous spans, the bottom flanges adjacent to intermediate supports are in compression and, if not sufficiently restrained, are prone to buckling. The buckling resistance thus depends on the restraint that is provided.

6.1 Beams without discrete bracing

Where there is no discrete bracing to the bottom flanges, the beams can buckle in a distortional mode, as shown in Figure 6.1. The restraint to this form of buckling is the continuous inverted U-frame action of the slab and the beam webs. It is possible to derive an effective slenderness for this situation, based on a beam on elastic foundation analogy, but that is too complex for a student project. For student projects, discrete bracing should be provided.

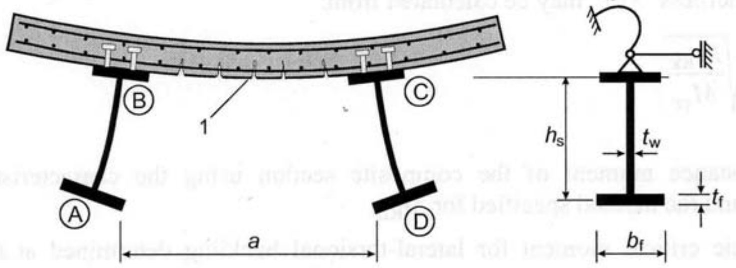


Figure 6.1 *Distortional buckling*

6.2 Beams with effective discrete bracing

In multi-girder decks, effective restraint can be provided by triangulated planes of bracing between pairs of beams, as shown in Figure 6.2. To be fully effective, the stiffness of the bracing needs to meet the stiffness criterion given in Section 10.1 of Part C but normally this is easily achieved and there need not be an explicit check in simplified designs.

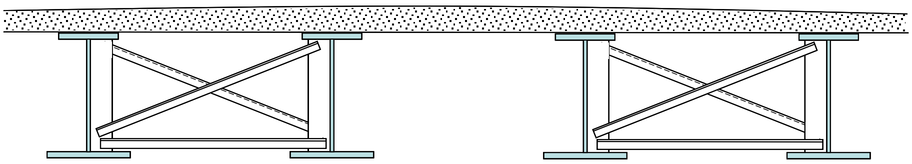


Figure 6.2 *Bracing between paired beams*

With fully effective bracing, the non-dimensional slenderness can be evaluated using the simple rule in Section 9.1 of Part B. This model treats the compression flange and part of the web as a compression strut, spanning between restraint positions, and this is a very convenient model for composite beams.

6.3 Beams with discrete bracing that is not fully effective

In ladder deck bridges, the U-frames formed by the composite cross girders and the web stiffeners provide a lateral restraint that is not fully effective. The buckling mode with this flexible restraint is similar to that shown in Figure 6.1 - the difference is that the discrete U-frames are much stiffer than the continuous U-frame action in the unbraced configuration. The half wavelength of buckling is now not constrained to the length between restraint positions (as it is with fully effective restraints) but extends over several cross girder spacings. An effective half wavelength and buckling load can be derived using a beam on elastic foundation model, as for the unbraced configuration, but is too complex for simplified design.

Instead, for the purposes of a student project, the slenderness may be determined using the simple rule in Section 9.1 of Part B but this time using double the cross beam spacing instead of the distance between points of restraint.

7 LONGITUDINAL SHEAR CONNECTION

To ensure composite action between the steel beam and concrete, the interface must be able to resist the longitudinal shear flow (calculated on the elastic section using bending theory). This is achieved by welding shear studs (see Figure 7.2) onto the top flange of the beam.

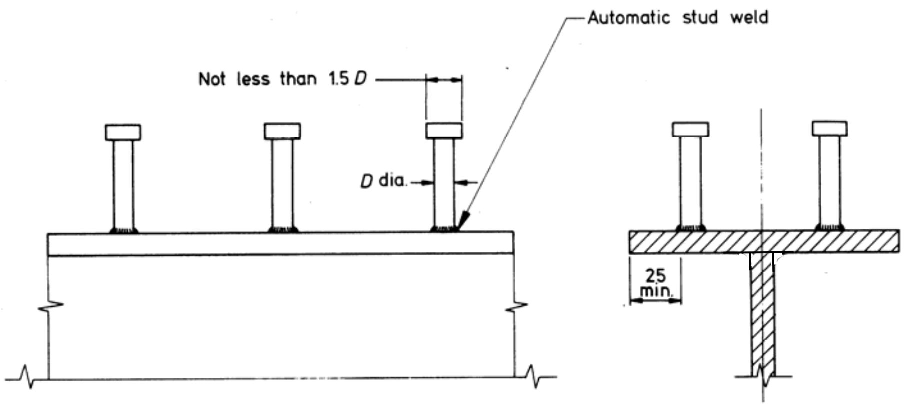


Figure 7.2 *Stud connector*

Studs are provided two or three in a row across the flange at between 100 mm and 300 mm centres. The number and spacing must be sufficient to provide a resistance of at least the calculated shear flow.

For a beam that is designed elastically, the shear flow between girder and slab varies along the beam, in proportion to the vertical shear at any position and the design value of the longitudinal shear is given by the following expression:

$$F_{\text{long}} = V_{\text{Ed}} \frac{A\bar{z}}{I_{\text{y}}} \tag{7.1}$$

Where

A	is the area of the slab (using the transformed area in 'steel units')
$\bar{z}$	is the height of the centroid of the slab about the y-y neutral axis
I_{y}	is the second moment of area of the composite section about the y-y axis
V_{Ed}	is the vertical shear at the cross section considered.

Note that uncracked section properties should be used, even where the slab is in tension, as this gives appropriate values of shear flow.

If plastic bending resistance is used, a different procedure is needed for more precise calculation of shear flow in the regions where M_{Ed} exceeds the elastic design resistance but for simplified design the above expression will give sufficiently accurate values.

The design resistance of a headed stud shear connector is given by expressions for the resistance of the material of the stud and for the resistance of the concrete in which the stud is embedded (the lower value of the two is used). For grade C40/50 concrete and for 19 mm diameter studs at least 100 high the resistance is governed by the steel strength and the value is:

P_{Rd} = 113 kN / γ_{V} = 90 kN

The designer must choose the number of studs in a row across the width of the flange and the spacing between the rows, to provide sufficient resistance for the design values of shear. The spacing of the rows is usually kept constant, either over the full length of the girder or, more economically over part lengths -

closely spaced rows will be provided for a length either side of support, widely spaced rows will be provided over a length in midspan and an intermediate spacing will be used for the other lengths. This variation reflects the different levels of shear in the different regions.

Stud spacing should not exceed 800 mm longitudinally.

8 REINFORCED CONCRETE SLAB

Design of the slab itself is considered outside the scope of the simplified bridge design covered by this and the related publications.

For illustrative purposes, if needed, the slab reinforcement may be taken as shown in Figure 8.2, provided that the slab does not span more than 4 m. For multi-girder decks this is a transverse section, i.e. the 20 mm bars are transverse to the main girders. For ladder decks the 20 mm bars are parallel to the main girders (in practice there are some detailing issues with the transverse reinforcement over the main girders but this need not be considered in student projects). For some configurations, extra longitudinal top reinforcement might be required over intermediate supports - this can be achieved by closing the spacing to 100 mm.

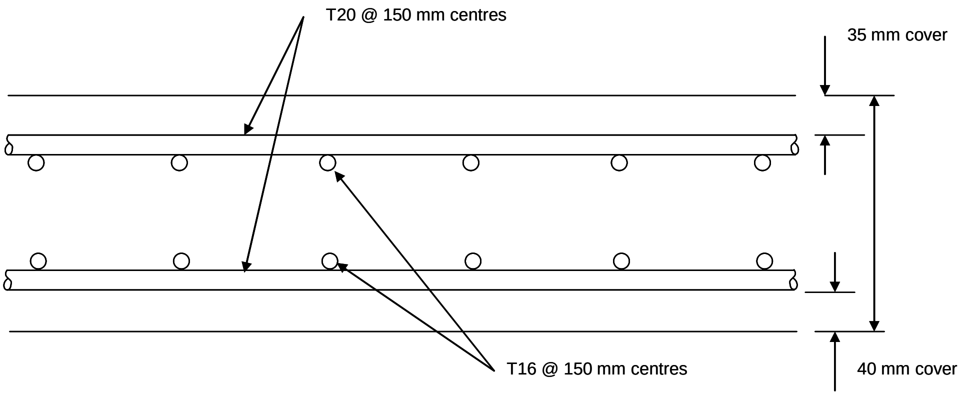


Figure 8.2 *Slab reinforcement*

9 CONTROL OF CRACKING

EN 1992-2 provides rules to control cracking at SLS. In most cases the aim is to ensure that calculated crack widths do not exceed 0.3 mm. Actual crack widths and the spacing of the cracks cannot be determined, because of the random nature of cracking patterns but expressions for design values can be given.

Two options are given - the use of bar sizes and spacings that are deemed to satisfy the requirements and the calculation of a crack width value.

For simplified design , it may be assumed that bars at least 16 mm diameter, at a spacing not exceeding 150 mm, will be adequate for control of cracking.